VOL. XXXVIII. No. 9 SEPTEMBER, 1960

THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

A Direct Solution of the Torsional-Flexural Buckling of Axially Loaded Thin-Walled Bars
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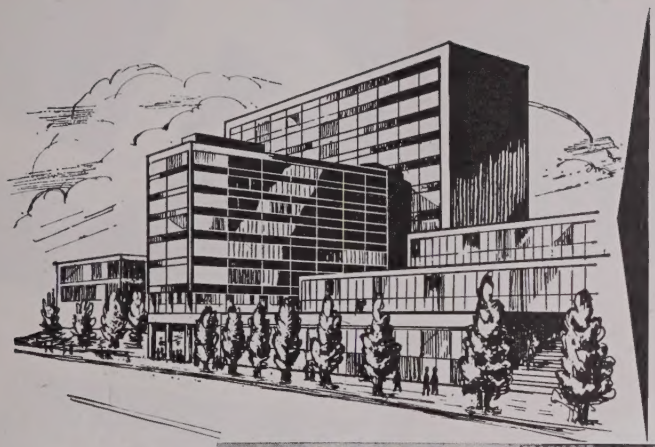

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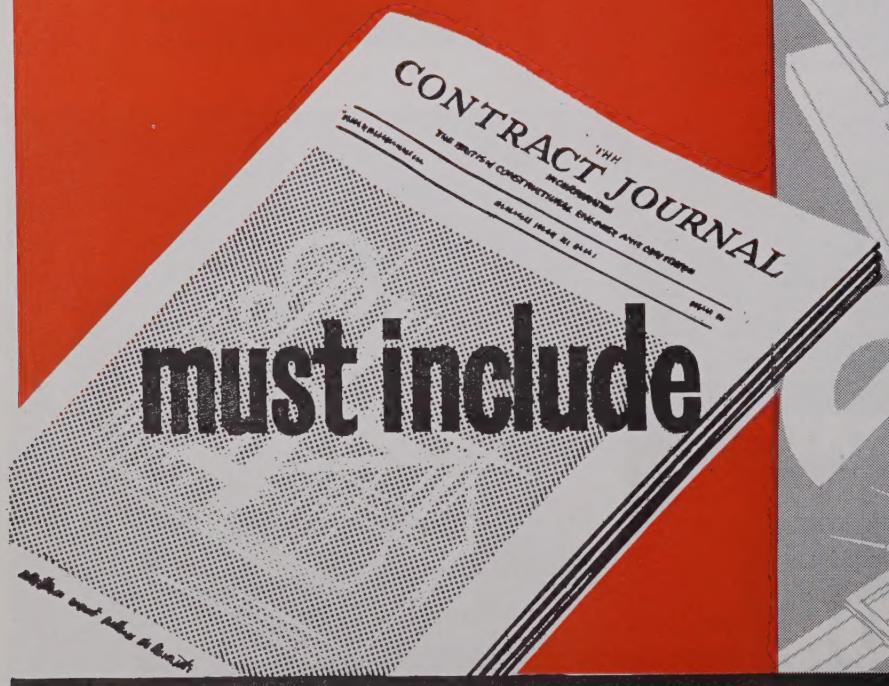

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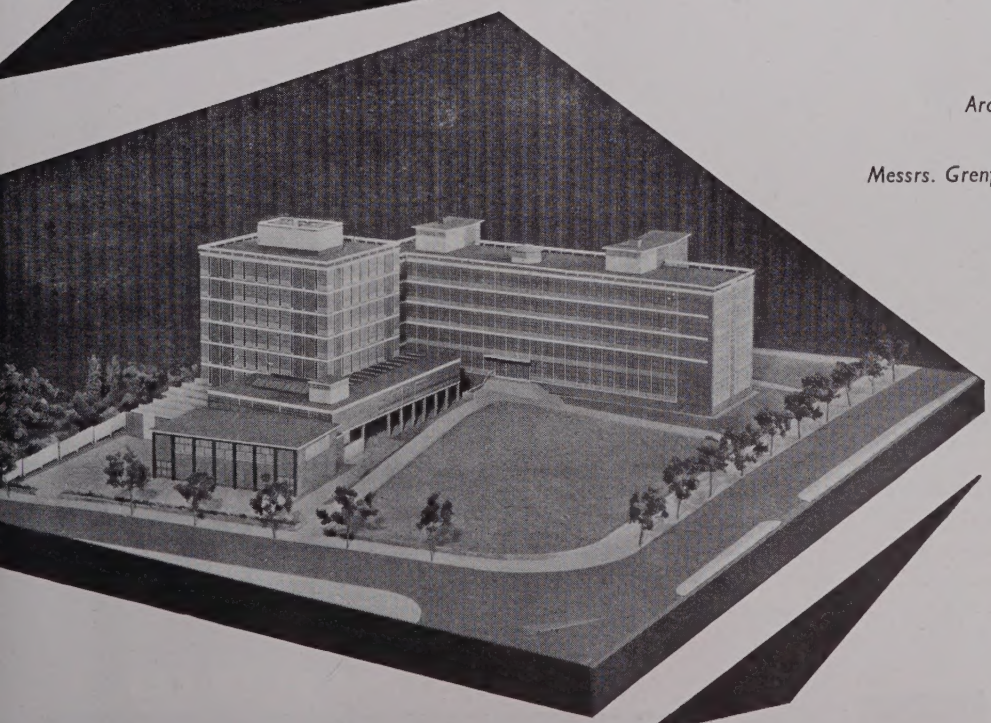

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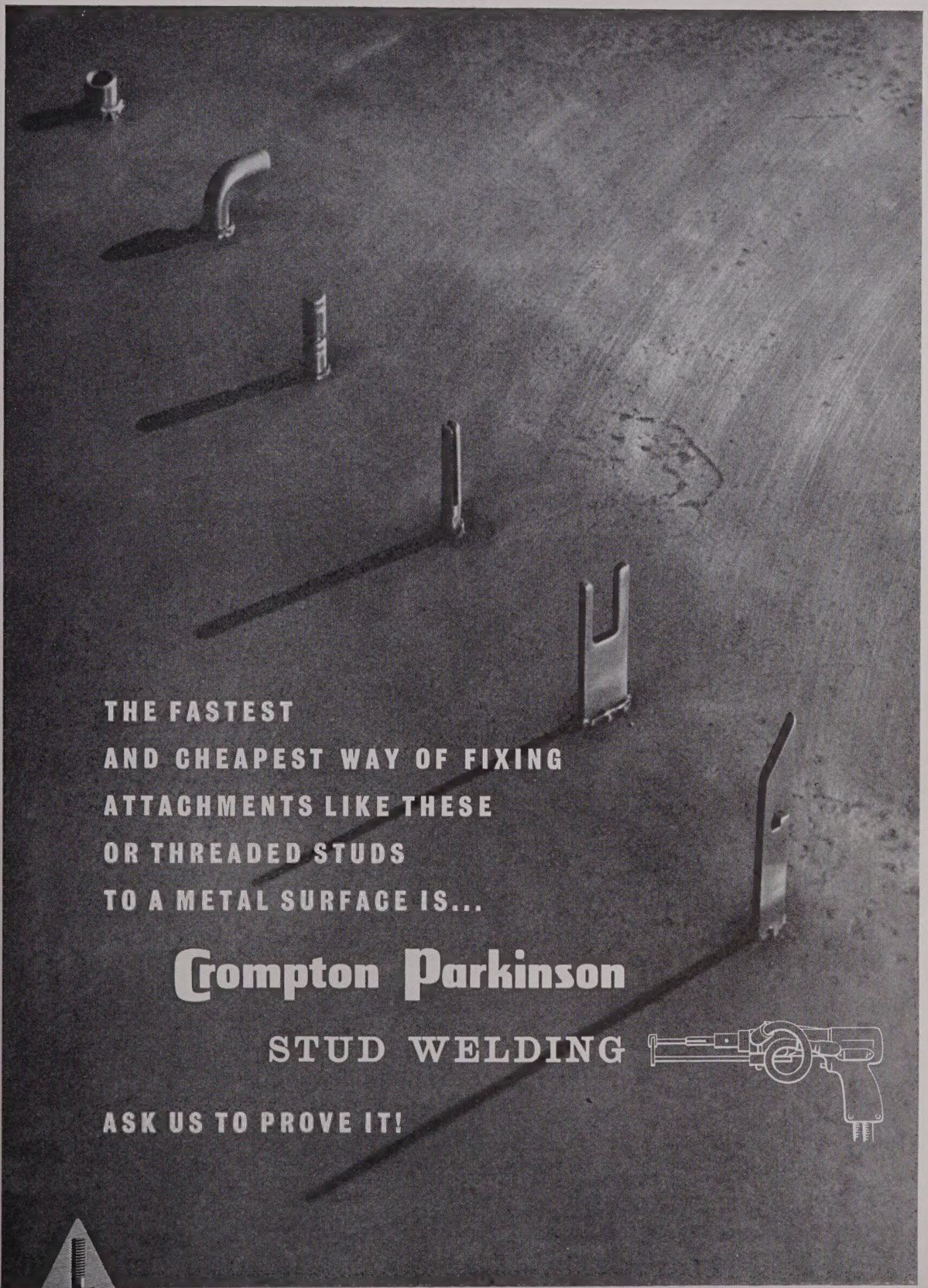
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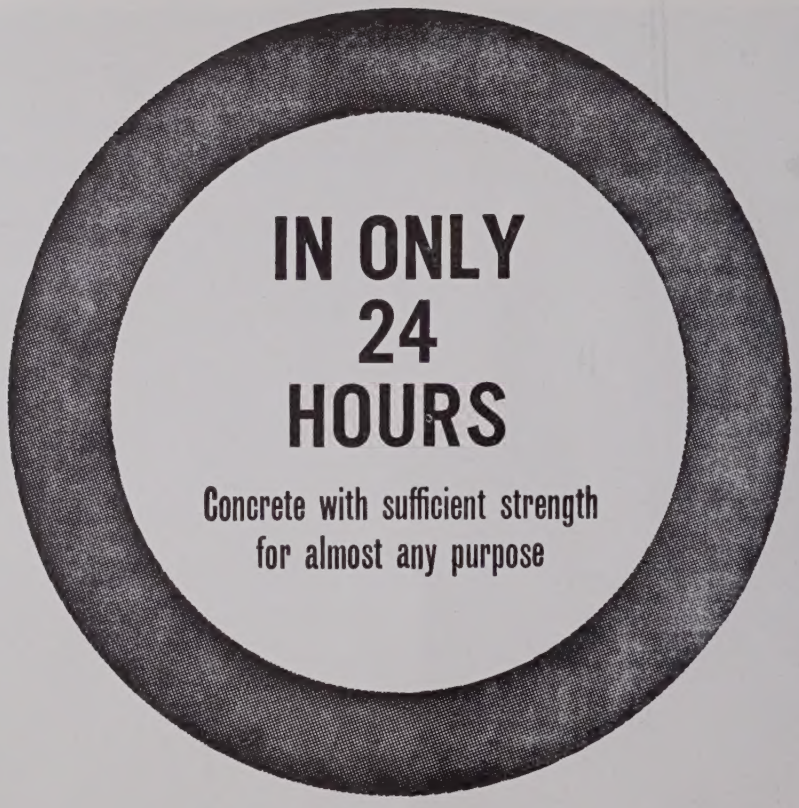

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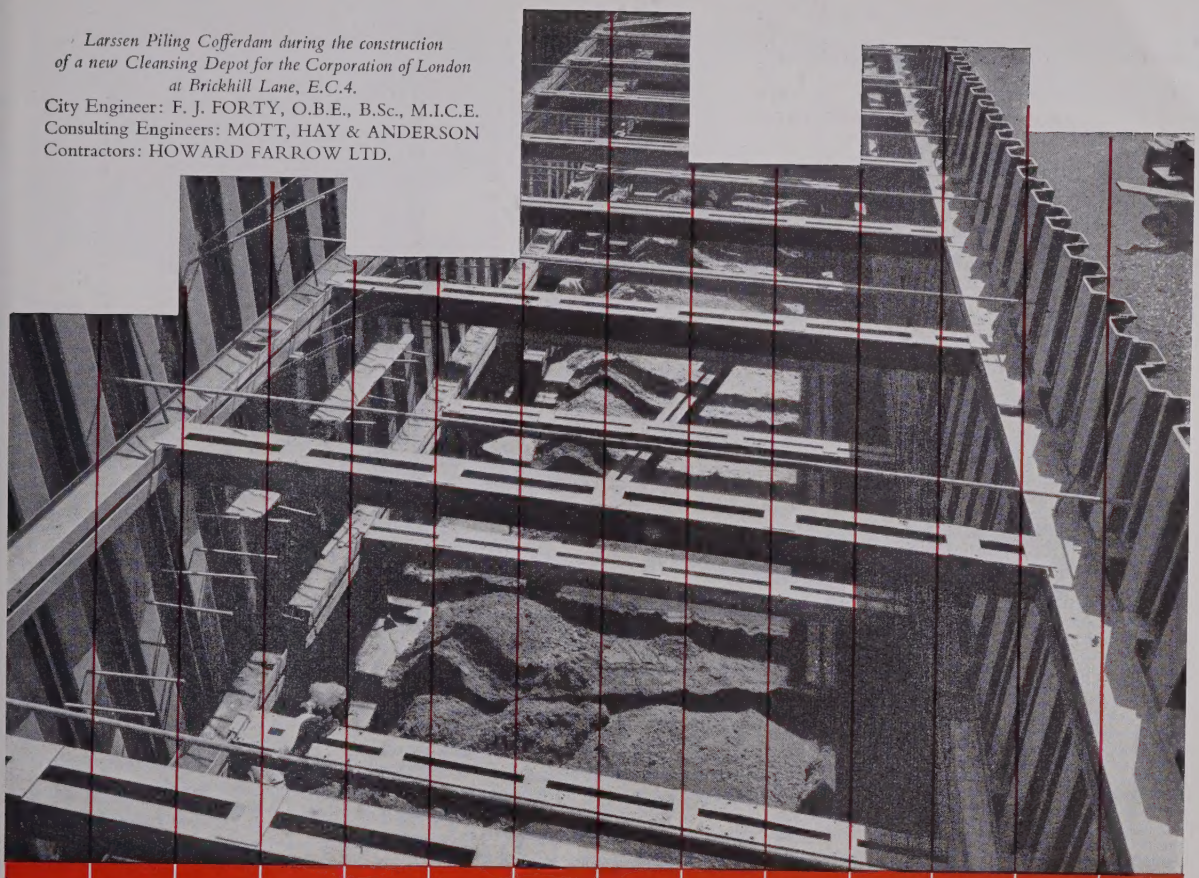

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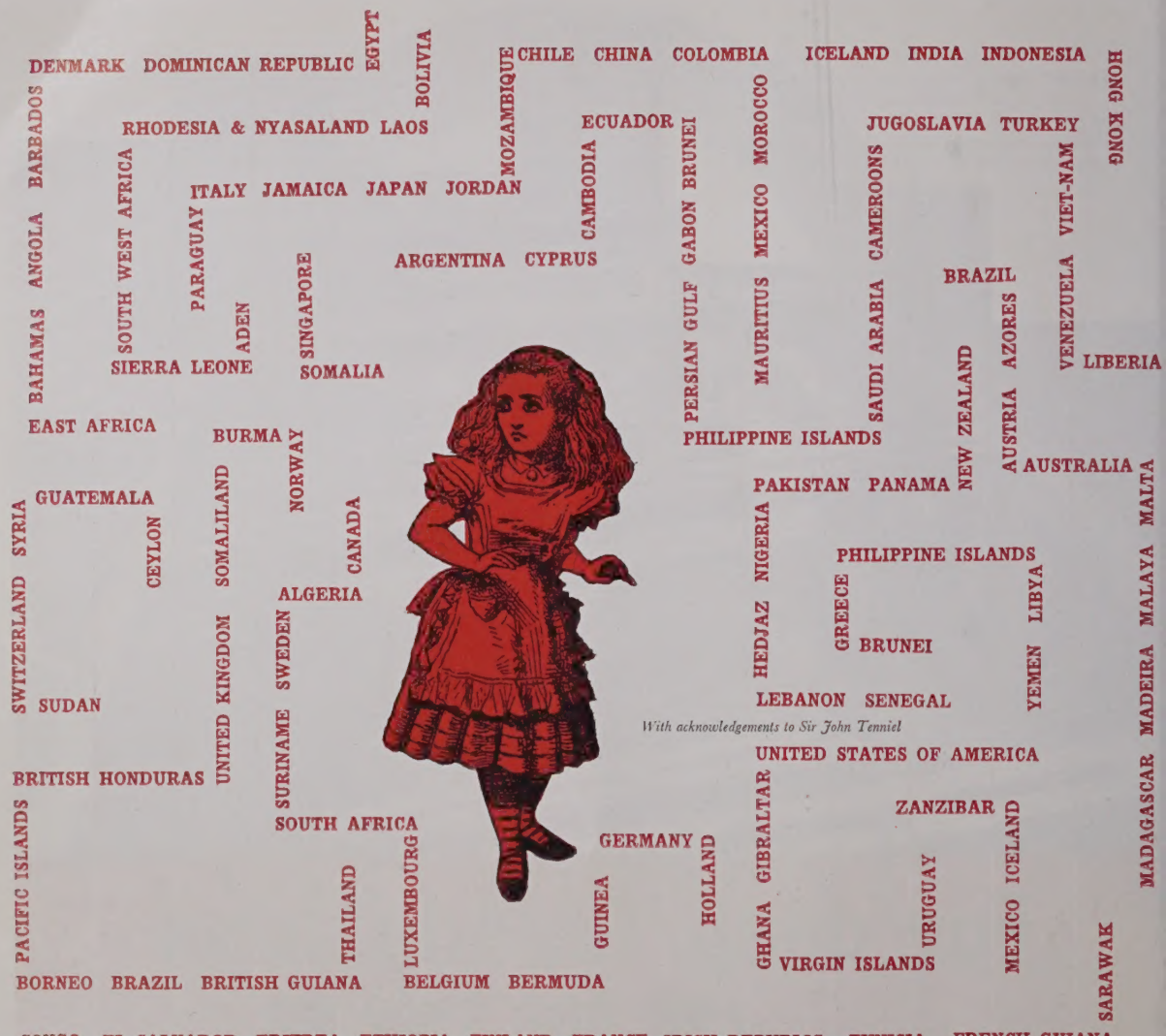
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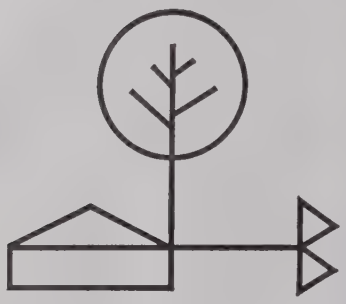

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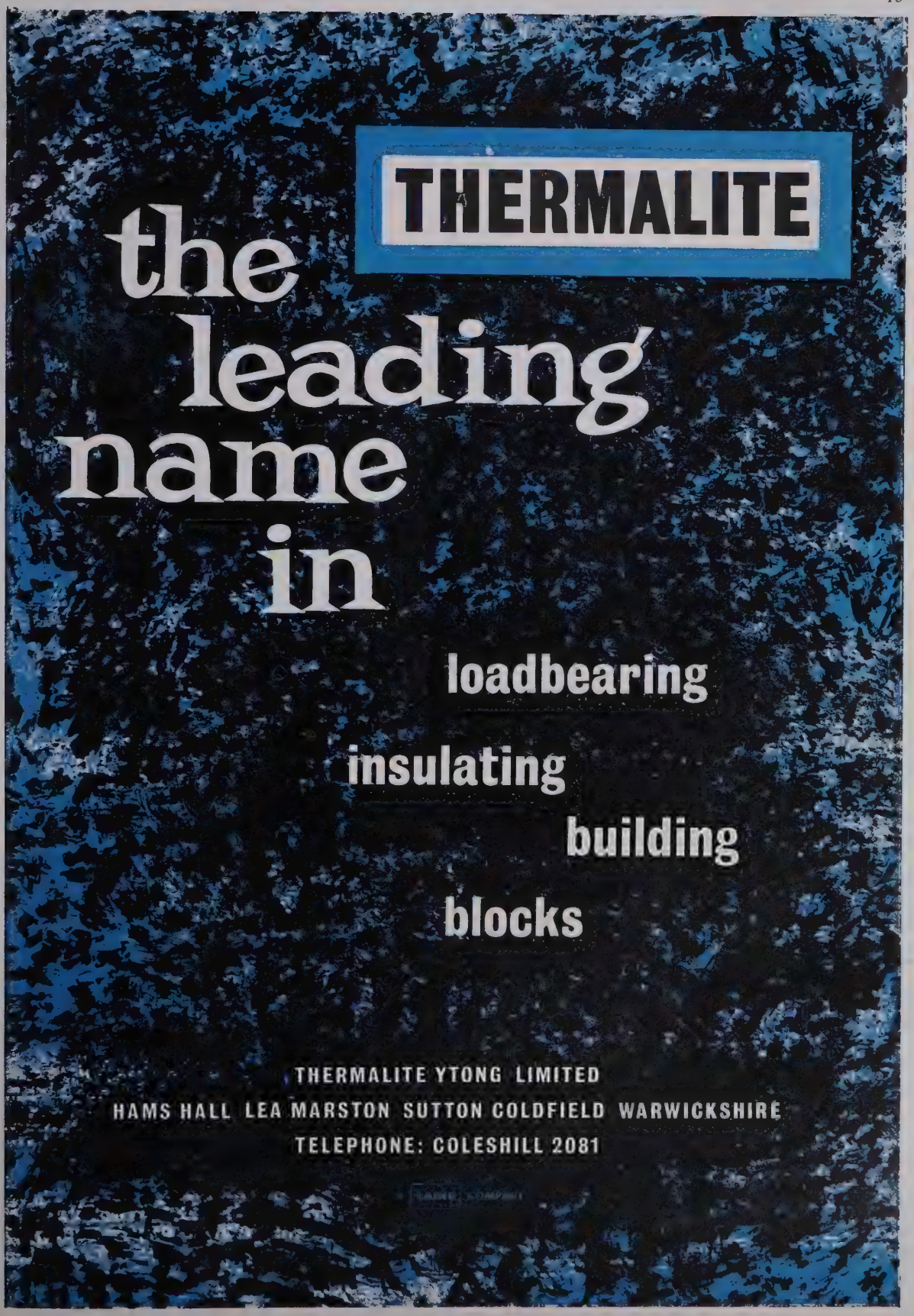
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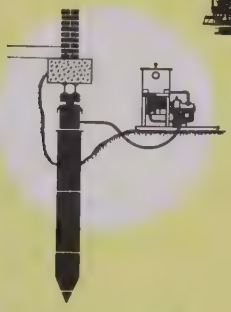

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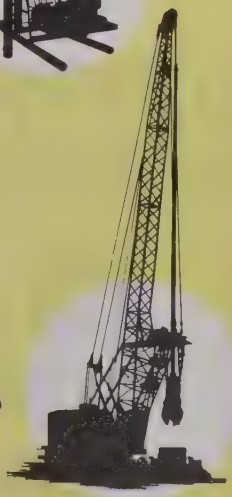

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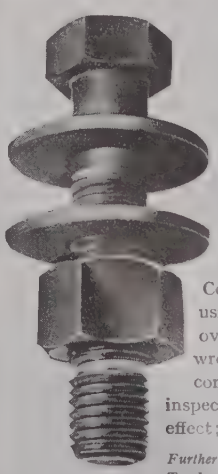

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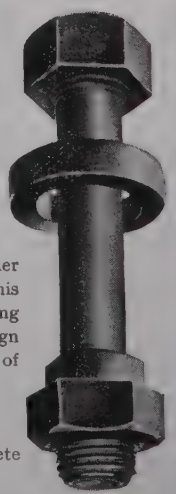

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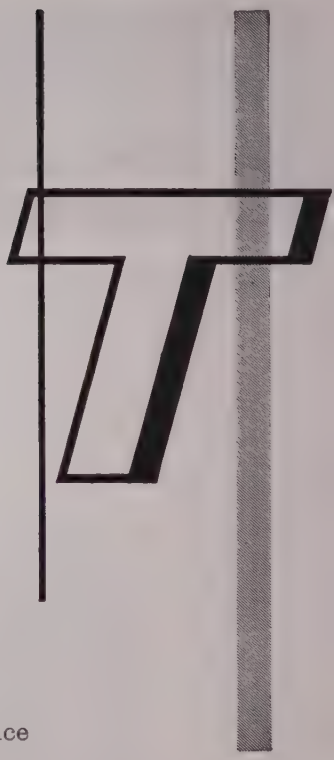

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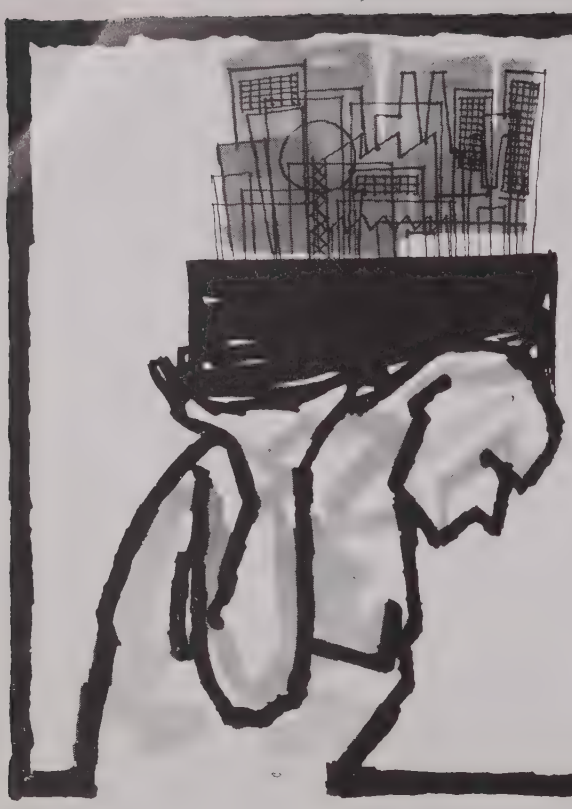

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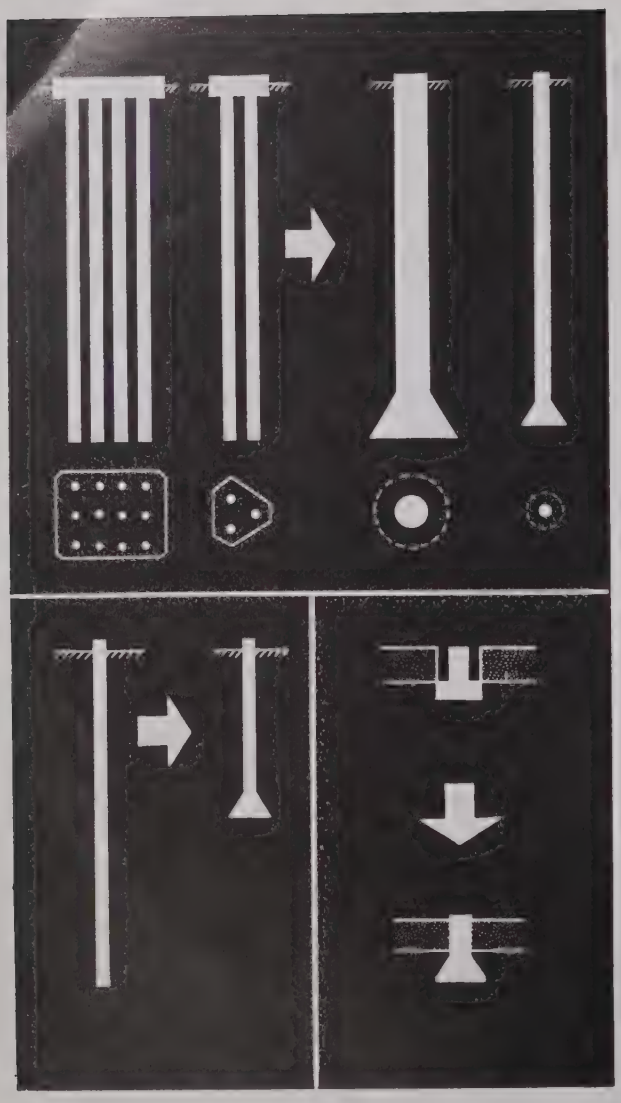

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A Direct Solution of the Torsional-Flexural Buckling of Axially Loaded Thin-Walled Bars

by J. D. Renton, B.Sc.

Engineering Department, University of Cambridge

Summary

A general solution of the equations for the torsional-flexural buckling of struts is presented. Expressions for the end conditions of the struts are found and their application to the buckling of frameworks described.

Notation

u_o — displacement of the shear centre axis in the x direction.

v_o — displacement of the shear centre axis in the y direction.

θ — the angle of rotation of the section about the z axis.

All differentiation is with regard to z.

x, y, z — the principal and longitudinal axes of the member.

I_x — the moment of inertia of the section about the principal axis $x-x$.

I_y — the moment of inertia of the section about the principal axis $y-y$.

E — Young's modulus.

GJ — the pure torsional stiffness of the section.

EK — the warping stiffness of the section.

(x_o, y_o) — the co-ordinates of the shear centre with regard to the centroid.

A — the area of the cross-section.

$r_o^2 = \dfrac{I_x + I_y}{A} + x_o^2 + y_o^2$.

Timoshenko's [1] sign convention is followed throughout this paper.

The values of P and σ, (the axial force and resulting stress) are positive for compression and negative for tension.

Introduction

The general Eulerian equations of the small deflection theory for the torsional-flexural buckling of thin-walled bars have been solved satisfactorily for certain types of members and end fixity[1,2]. However only approximate solutions exist for the remaining cases[3].

In the present analysis the Eulerian equations are solved and the end conditions expressed in geometrical terms. This enables an exact solution to be made in all cases where the present theory is applicable.

I. The Solution of the Eulerian Equations

The Eulerian equations may be written [2, 4]

$$EI_y u_o'''' + P(u_o'' + y_o\theta'') = 0, \quad (1)$$

$$EI_x v_o'''' + P(v_o'' - x_o\theta'') = 0, \quad (2)$$

$$GJ\theta'' - EK\theta'''' - P(y_o u_o'' - x_o v_o'' + r_o^2\theta'') = 0, \quad (3)$$

where primes indicate differentiation with respect to z.

From these equations

$$D^6[(EI_xD^2+P)(EI_yD^2+P)\{(GJ-Pr_o^2)-EKD^2\} - Px_o^2(EI_yD^2+P)+Py_o^2(EI_xD^2+P)](u_o, v_o \text{ or } \theta) = 0, \quad (4)$$

where D is the operator (d/dz).

Equations (1) to (3) must hold continuously throughout the range of z. Therefore the coefficients of each independent function of z in these equations must be zero. The expressions for (u_o, v_o and θ) resulting from equation (4) may be substituted in equations (1) to (3). Then applying the above condition the final form of these expressions becomes

$$u_o = \Sigma[U_{an}\,exp.(q_n z) + U_{bn}\,exp.(-q_n z)] + \alpha_1 z + \alpha_o, \quad (5)$$

$$v_o = \Sigma[V_{an}\,exp.(q_n z) + V_{bn}\,exp.(-q_n z)] + \beta_1 z + \beta_o, \quad (6)$$

$$\theta = \Sigma[\Theta_{an}\,exp.(q_n z) + \Theta_{bn}\,exp.(-q_n z)] + \gamma_1 z + \gamma_o, \quad (7)$$

for the range n = 1 to 3, where

$$U_{(a\text{ or }b)n} = \frac{-Py_o\Theta_{(a\text{ or }b)n}}{EI_y q_n^2 + P}, \qquad V_{(a\text{ or }b)n} = \frac{Px_o\Theta_{(a\text{ or }b)n}}{EI_x q_n^2 + P}, \quad (8)$$

Θ_{an}, Θ_{bn}, α_1, α_o, β_1, β_o, γ_1, γ_o are constant coefficients which may be evaluated from the end conditions, and

$$\begin{vmatrix} EI_y q_n^2 + P & 0 & Py_o \\ 0 & EI_x q_n^2 + P & -Px_o \\ -Py_o & Px_o & GJ - EKq_n^2 - Pr_o^2 \end{vmatrix} = 0 \quad (9)$$

It will be seen that equation (9) is a discriminating cubic in q_n^2; thus q_n may be real or imaginary, but not complex.

II. End Conditions

In his paper on corrected discontinuities, Chilver[4] considers any element of a strut to be acted upon by two systems of equilibrating lateral forces. The first system (S_1) is the equilibrant of the end load and the second system (S_2) equilibrates the moments producing the distorted form. (See figure I). The lateral displacement of any element with co-ordinates (x, y) is approximately $\{u_o + (y_o - y)\,\theta,\ v_o - (x_o - x)\,\theta\}$. Then

$$S_{x1}\,\delta z = -\sigma t ds \cdot \frac{d}{dz}\{u_o + (y_o - y)\,\theta\}\,\delta z,$$

$$S_{y1}\,\delta z = -\sigma t ds \cdot \frac{d}{dz}\{v_o - (x_o - x)\,\theta\}\,\delta z,$$

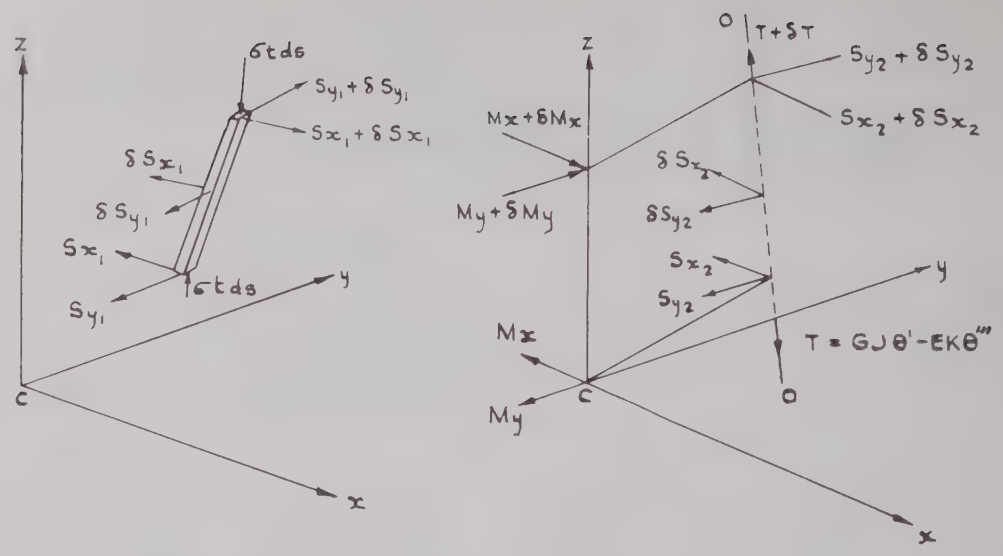
$$S_{x2}\,\delta z = -(M_y)'\,\delta z = -EI_y u_o'''\,\delta z,$$

$$S_{y2}\,\delta z = (M_x)'\,\delta z = -EI_x v_o'''\,\delta z.$$

The total shear forces F_x, F_y in the x, y, directions respectively are

$$F_x = \int_A (S_{x1} + S_{x2}) = -[P(u_o' + y_o\theta') + EI_y u_o'''] \quad (10)$$

$$F_y = \int_A (S_{y1} + S_{y2}) = -[P(v_o' - x_o\theta') + EI_x v_o'''] \quad (11)$$



I(a) First System I(b) Second System

cz Direction of Centroidal axis

oo Direction of Shear Centre axis

Fig. I.

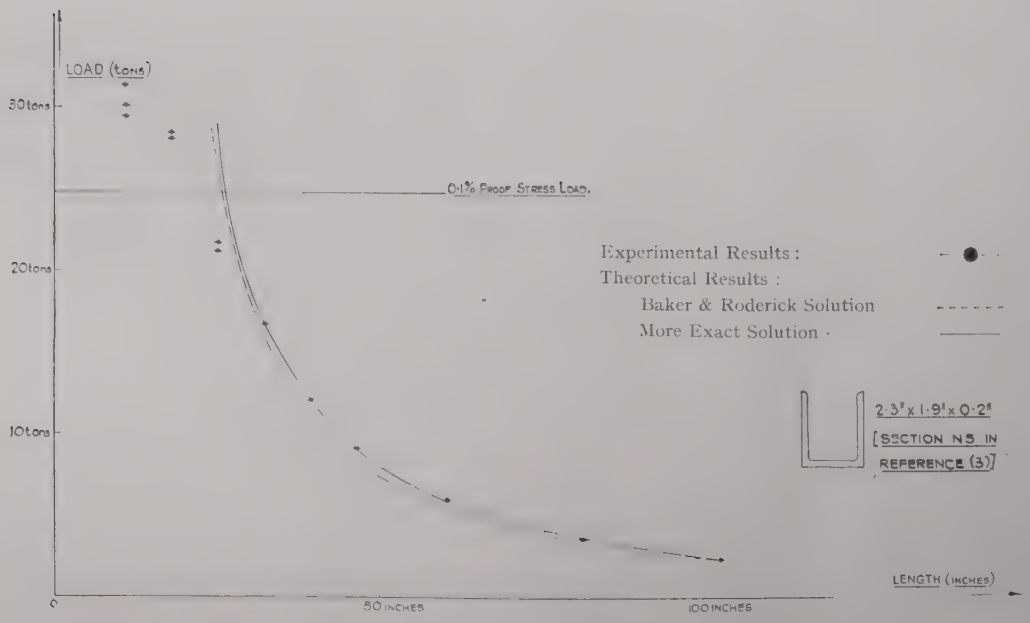


Fig. II—Torsional-flexural buckling of a light-alloy channel section

The total torque T is the sum of that required to produce the twisted form and the couples produced by the shear forces about the shear centre axis

$$T = GJ\,\theta' - EK\,\theta''' + \int_A S_{x1}(y_o - y) - \int_A S_y{}^1(x_o - x),$$

which becomes

$$T = GJ\,\theta' - EK\,\theta''' - P(y_o u_o' - x_o v_o' + r_o{}^2\,\theta'). \quad (12)$$

Integrating equations (1) to (3) indicates that F_x, F_y and T are constant along the member.

Provided that P continues to act through the centroid at the end cross-sections, then at each end

$$M_y = EI_y u_o'', \quad (13)$$

$$M_x = -EI_x v_o''. \quad (14)$$

A further system of end loading known as a bi-moment[5] (B_z) may occur. This is produced by equal and opposite couples acting about any given axis perpendicular to the longitudinal axis of the member. The bi-moment as defined by Vlasov may be expressed by the equation

$$B_z = -EK\,\theta''. \quad (15)$$

Any permissible form of boundary condition may now be expressed in geometrical terms. The boundary conditions together with equations (5) to (9) are then sufficient for the solution of a problem.

III. A Solution using the above Equations compared with a previous approximate solution

Baker and Roderick[3] have tested aluminium alloy struts rigidly attached at each end to universal ball joint fittings. They use Timoshenko's[1] equation.

$$(x_o{}^2 + y_o{}^2 - r_o{}^2)P^3 + [(P_1 + P_2 + P_3)r_o{}^2 - x_o{}^2P_1 - y_o{}^2P_2]\,P^2 - r_o{}^2\,(P_1P_2 + P_2P_3 + P_3P_1)\,P + P_1P_2P_3\,r_o{}^2 = 0, \quad (16)$$

and take

$$P_1 = \frac{\pi^2 EI_y}{l^2},\ P_2 = \frac{\pi^2 EI_x}{l^2},\ P_3 = \frac{1}{r_o{}^2}\left(GJ + EK\,\frac{4\pi^2}{l^2}\right) \quad (17)$$

P_1 and P_2 are the Euler loads, and P_3 is the fixed end torsional buckling load. However Timoshenko's equation is only strictly true for compatible end conditions. Also P_3 is derived on the assumption that both warping and rotation about the z axis are prevented at the ends.

In the tests carried out by Baker and Roderick all the struts tested had at least one axis of symmetry. If it is assumed that at the ends of the struts the bending and twisting moments are zero, and that the displacement of the centroid and warping are prevented, then equations (5) to (9) and (12) yield the following equations.

$$\cosh q_1 l/2 = 0 \left(q_1 = i\sqrt{\frac{P}{EI_y}}\right), \quad (18)$$

$$\frac{1}{q_2}\left(q_2{}^2 + \frac{P}{EI_x}\right)\tanh q_2 l/2 = \frac{1}{q_3}\left(q_3{}^2 + \frac{P}{EI_x}\right)\tanh q_3 l/2, \quad (19)$$

$$\frac{1}{q_2}\left(q_2{}^2 + \frac{P}{EI_x}\right)\coth q_2 l/2 = \frac{1}{q_3}\left(q_3{}^2 + \frac{P}{EI_x}\right)\coth q_3 l/2, \quad (20)$$

where q_n ($n = 1, 2$ or 3) are the roots of equation (9).

The lowest critical loads from the two theories are compared in Fig. II. This graph shows the results for channel section N.S. tested by Baker and Roderick[3].

It is also possible to allow for the size of the end fittings. The equations corresponding to equations (18) to (20) are then

$$\tan\sqrt{\frac{P}{EI_y}}\,\frac{l}{2} = \frac{1}{R\sqrt{P/EI_y}}, \quad (21)$$

$$\frac{1}{EI_x q_2{}^2 + P}\left[EI_x q_2 \coth q_2 l/2 - PR\right] = \frac{1}{EI_x q_3{}^2 + P}\left[EI_x q_3 \coth q_3 l/2 - PR\right], \quad (22)$$

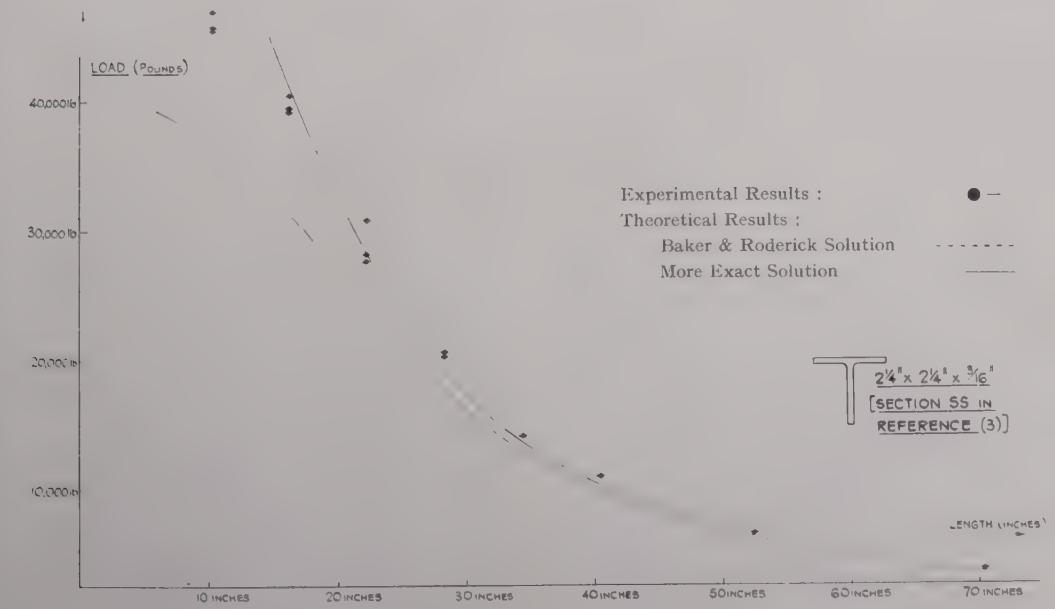


Fig. III—Torsional-flexural buckling of a light-alloy tee section

$$\frac{1}{q_2\,(EI_x q_2^2 + P)}\left[\left\{2PR + EI_x q_2^2 \frac{Px_0^2 l(l+2R)}{GJ - Pr_0^2}\right\} tanh\, q_2 l/2 - \left\{2PR^2 q_2 + PRq_2 \frac{Px_0^2 l(l+2R)}{GJ - Pr_0^2}\right\}\right]$$
$$= \frac{1}{q_3\,(EI_x q_3^2 + P)}\left[\left\{2PR + EI_x q_3^2 \frac{Px_0^2 l\,(l+2R)}{GJ - Pr_0^2}\right\} tanh\, q_3 l/2 - \left\{2PR^2 q_3 + PRq_3 \frac{Px_0^2 l\,(l+2R)}{GJ - Pr_0^2}\right\}\right], \quad . \quad . \quad . \quad (23)$$

where R is the distance from the end of the member to the centre of the universal ball joint about which it rotates. Equations (21) to (23) are used to find the lowest critical load for the tee section *SS* tested by the same authors. The results are compared in Fig. III.

IV. Frameworks

The above theory may also be applied to the buckling of frameworks. Tests have been made with ' Xylonite ' models and the experimental results agree well with the theory. The calculations are simplified by assuming either infinite or zero warping restraints at the ends of the members. At any free joint the remaining end conditions may be found from the equilibrium of forces and moments and the compatibility of joint displacements and rotations. This results in a set of simultaneous equations from which the critical load may be evaluated.

Except in the case of simple frameworks the solution of the above equations is too complex for manual calculation. However a computer programme based on one written by Livesley[6] has been used to evaluate the theoretical results for the above model tests. This programme is applicable to plane frameworks composed of members with bi-symmetrical sections or sections with one axis of symmetry and no warping stiffness.

Conclusion

The solution of the Eulerian equations and geometrical expressions for the end conditions of a strut have been presented. Their application to certain problems has been described and a comparison made with an approximate theory. Though the calculations involved are usually more difficult than those for the approximate theories, it is felt that the more exact results may justify its use in certain cases.

Acknowledgements

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by November 30th, 1960.

Book Review

A Source Book in Mathematics, Vols. I and II. Edited by David Eugene Smith. (New York : Dover Publications Inc. ; London : Constable, 1960) 8 in. × 5⅜ in., 701 + xiii pp. and 701 + xiii pp. 28s.

This new edition of an anthology of mathematical writings, first published in 1929, is an unabridged and unaltered republication in two volumes in an inexpensive paperback edition. The work presents, in English translation, the great discoveries in mathematics from the Renaissance to the end of the nineteenth century. It contains selections from one hundred and twenty-five treatises and articles taken from the writings of all the major historical figures, Euclid, Descartes, Gauss, Galileo, Newton, Leibnitz, Pascal, Riemann, Bernoulli and others and also from many interesting but relatively little known mathematicians.

Volume I considers the history of mathematics chronologically from primitive concepts of counting to the great schools of Egypt, Babylonia, early and late Greece, the development of oriental mathematics, the Roman and medieval worlds, and up to the end of the 19th century. Volume II deals with specific fields and problems and indicates the gradual advance of arithmetic, geometry, algebra, trigonometry, calculus, calculating machines etc.

Each article is preceded by a biographical introduction and footnotes supply many technical points outside the scope of the book and refer to disputed matters and original or supplementary sources.

Hundreds of fine illustrations drawn from Egyptian papyri, Hindu, Chinese and Japanese manuscripts, Greek, Roman and medieval texts, maps, portraits, etc. are included in the volumes, as well as many modern diagrams.

The Stability of Beams with Buckled Compression Flanges

by S. Cherry *

Summary

A method is proposed for estimating the elastic lateral instability load of beams taking into account the possibility of premature compression flange failure by local buckling. This introduces the principle of a hypothetical or transformed section which is made locally stable at every stage of the loading through the application of the concept of effective compression plate width. Using a relation for section transformation developed from the experimental results in the light of relevant theoretical work, it was found that the procedure suggested gave reasonable estimates for the critical loads of beams in which there is an interaction between the local and lateral buckling modes.

Notation

A — primary flexural rigidity.
B — secondary flexural rigidity.
C — torsional rigidity.
C_w — warping rigidity.
E — Young's modulus of elasticity.
G — shear modulus of elasticity.
L — span length.
M_{cr} — critical moment for lateral instability.
P_1, P_3 — critical loads for Euler and pure torsional buckling respectively for beam sections loaded as columns.
b, t — width and thickness respectively of compression flange in original section.
$\bar{b}$ — width of compression flange in transformed section.
f — plate function for effective width reductions.
β_1 — geometric constant of cross section.
γ — $\left(1 - \frac{B}{A}\right)$
σ_{cr} — stress for local flange buckling.
σ_e — stress at web-flange junction.

Certain additional symbols are defined where first introduced in the text.

1. Introduction

The search for and development of more efficient structural forms has resulted in the wider use of thin-walled sections in which characteristic instability problems feature prominently in design. The general theory regarding the elastic instability of beams whose cross section shapes remain undistorted during loading is well known and the critical load may be obtained by standard solutions. However, existing theory does not take into account the effect of premature local plate buckling of thin outstanding compression flanges on the overall collapse load. Apart from purely academic considerations, practical interest in this problem has recently been created as a result of the development of a new enthusiasm for deep welded plate girders in which the interaction of local and lateral instability may be relevant.

The work reported in this paper was originally undertaken to contribute towards an understanding of the interaction between local and lateral instability. The investigation is concerned primarily with the behaviour of thin-walled beams subjected to loads in the range between the load first producing local elastic buckling of the compression flanges and the load causing overall elastic lateral instability, subsequently referred to as the 'elastic post-buckling range.' The study is confined to the case of bisymmetrical and monosymmetrical I-beams subjected to pure terminal couples acting in the plane of the web. Only those sections in which the compression flanges alone buckle are considered; the webs are assumed to be sufficiently stiff so as not to undergo local distortion. However, it would appear that the principle suggested is fairly general and could be extended to cover other cases.

The paper is divided into two parts. In Part I a method is suggested for calculating the buckling strength of beams resulting from an interaction of local and lateral instability. Part II is devoted to a description of the testing programme and to a comparison between the calculated and experimental results. Some general conclusions are also drawn.

PART 1

2. Theory of Beam Buckling

2.1 *Monosymmetrical I-Beams of Undeformed Section*

The criterion of instability for a beam bent in its plane of maximum flexural stiffness is normally expressed as the least load necessary to produce a bifurcation of the equilibrium position. The differential equations governing the buckling condition for a simply supported monosymmetrical I-beam subjected to circular bending in the plane of the web have been reported by Goodier[1] and Timoshenko[2]. In their derivations these authors assumed that the secondary flexural rigidity is small in comparison with the primary rigidity. This restriction may be avoided by introducing the geometrical relationships between displacement and curvature as first employed by Prandtl[3] and Michell[4]. With the stipulation that the beam is hinged about the principal axes but rigidly held against twist at the ends where warping is allowed to proceed unhindered the Prandtl-Michell approach leads to the following value for the moment corresponding to the critical state:

$$M_{cr} = \frac{P_1\beta_1}{2\gamma} \pm \sqrt{\frac{P_1^2\beta_1^2}{4\gamma^2} + \frac{P_1 P_3 r_o^2}{\gamma}} \quad \cdot \quad \cdot \quad (1)$$

* Assistant Professor, Department of Civil Engineering, University of British Columbia.

where $\gamma = \left(1 - \frac{B}{A}\right)$, B and A are the secondary and primary flexural rigidities respectively.

β_1 is a property of the cross section depending only on its geometry and defined in the Appendix.

r_o is the radius of gyration of the section with respect to its shear centre.

$P_1 = \frac{\pi^2 B}{L^2}$, the Euler load for column buckling of the section about its minimum principal axis.

$P_3 = \frac{1}{r_o^2}\left(C + \frac{\pi^2 C_w}{L^2}\right)$, the critical load for purely torsional column buckling of the section.

C and C_w are the torsional and warping rigidities respectively.

L is the span length.

Equation (1) is identical with the equivalent expression given in Timoshenko's paper with the addition of the γ term in the denominator. It is assumed that C is small in comparison with A.

The middle half of a fixed-ended beam is in the same condition as a pin-ended beam and therefore the solution for the former case evidently corresponds to the calculation made by (1) provided $L/2$ is substituted for L in the terms defining P_1 and P_3. Accordingly, for a fixed-ended beam the P-values to be used in (1) are :

$$P_1 = \frac{4\pi^2 B}{L^2}\ ;\ P_3 = \frac{1}{r_o^2}\left(C + \frac{4\pi^2 C_w}{L^2}\right) \quad \cdot \quad \cdot \quad (2)$$

The actual support conditions associated with the experimental phase of this work corresponded to simply supported ends with full restraint against warping. For such boundaries the bifurcation state is governed by a transcendental equation to which a trial and error procedure must be applied to yield the critical moment for instability. The use of the transcendental equation for treating the interacted buckling problem by the method subsequently proposed is tedious and unwarranted. A direct solution providing a reasonable estimate to the equation can be obtained by noting that the ends are hinged as regards bending but effectively built in as regards twisting. This suggests using (1) as an approximation to the transcendental solution for the critical moment but with P_1 and P_3 having the mixed values

$$P_1 = \frac{\pi^2 B}{L^2}\ ;\quad P_3 = \frac{1}{r_o^2}\left(C + \frac{4\pi^2 C_w}{L^2}\right) \quad \cdot \quad \cdot \quad (3)$$

to account for the dissimilar end conditions. Accordingly, the form of the approximate equation governing all subsequent work becomes :

$$M_{cr} = \frac{\pi^2 B \beta_1}{2\gamma L^2} \pm \sqrt{\frac{\pi^4 B^2 \beta_1^2}{4\gamma^2 L^4} + \frac{\pi^2 B}{\gamma L^2}\left(C + \frac{4\pi^2 C_w}{L^2}\right)} \quad (4)$$

Baker[5] and Roderick have obtained satisfactory results by applying this method of approach to strut tests and Flint[6], in effect, has analyzed plastic buckling of I-beams using the same principle.

2.2 *Treatment of Beams with Locally Buckled Compression Flanges*

(a) *Post-Buckling Behaviour and the Transformed Section*

When flange plate buckling precedes the primary instability condition the values of the rigidities upon which the predictions of the critical moment depend become a function of applied stress, decreasing with increasing stress due to the growth of the accompanying wave formations. While local failure does not necessarily pre-suppose immediate collapse of the parent section, the inception of the critical state of buckling is accelerated and beams become unstable laterally in combination with a local distortion of their cross sections at loads which are less than those given by (4). In discussing the occurrence of a combined instability involving both local waving of the compression flange and its lateral displacement the problem primarily, therefore, is to determine the rate at which the rigidities A,B,C, and C_w are reduced once the flange stresses due to bending about the major principal axis of the beam are sufficient to produce local buckling of the compression plates.

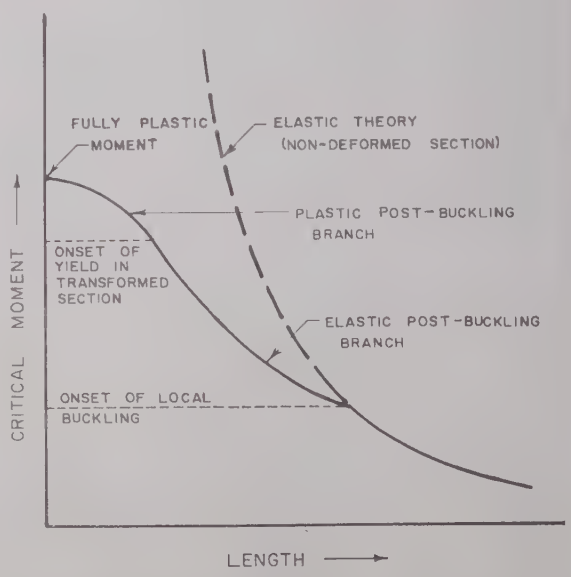

Fig. 1.—Diagram Illustrating Lateral Buckling Strength of Locally Post-Buckled Beams.

A qualitative picture showing the variation in buckling strength for a section suffering from local distortion is shown in Fig. 1. Between the onset of local instability and final overall collapse (the post-buckling range) the beam buckles elastically provided plastic deformations do not occur. When the yield stress at any point in the beam is exceeded the rate at which the effective beam rigidities are diminished is increased further and lateral collapse takes place with the section in a plastic post-buckled condition. The ultimate strength obtainable coincides with the fully plastic moment of the original undeformed beam. The studies which follow are mainly concerned with the elastic branch of the interaction curve.

The theoretical complexity of the post-buckling behaviour of plate assemblies precludes an analytical treatment for the evaluation of modified rigidity values corresponding to varying stress levels. However, it is possible to compensate continuously, and in a fairly simple manner, for the extent to which the values of the initial rigidities are reduced once the applied loads reach a magnitude sufficient to produce local flange buckling. This is conveniently accomplished

by introducing the concept of a transformed section through the use of the principle of effective width which has been well established for plate instability work involving post-buckling behaviour.

von Karman[7] first proposed the idea of equivalent width as a semi-empirical procedure to evaluate the ultimate load carrying capacity of single plates exhibiting a post-buckling strength. Later, Winter[8] established experimentally that this principle could similarly be applied to the compression elements representing component parts of structural members and was also valid at stresses below the yield point.

According to the basic hypothesis, the stress in the regions most heavily distorted by the local wave formations remains substantially constant at the critical value while the constrained portions of the plate continue to function and carry the increased stress above the critical. Then for beam sections of the type under discussion, at any stage of loading beyond the critical, the buckled flange of width b and the actual non-uniform stress distribution, Fig. 2, is replaced by a reduced width, $\bar{b}$, which is assumed fully effective and uniformly stressed to the maximum value of the actual distribution. The value of $\bar{b}$ is defined so that the total load carried by the actual and fictitious flange is identical.

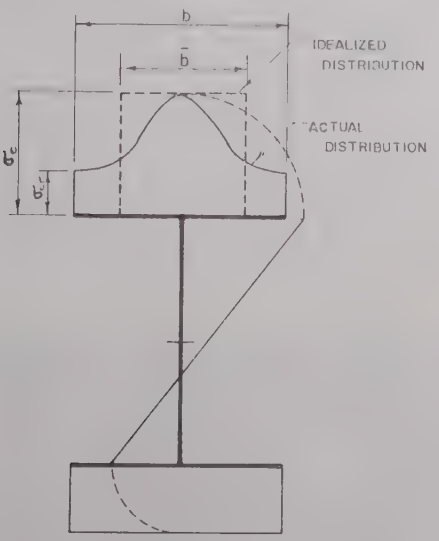

Fig. 2.—Stress Variations in Post-Buckled Flange.

Theoretical[9] and experimental[8] studies have indicated that $\bar{b}$ can be expressed as a function of σ_{cr}, the critical stress corresponding to the onset of local buckling, σ_e, the bending compressive stress at the web-flange junction, and b. This relationship may be written in non-dimensional form as

$$\left(\frac{\bar{b}}{b}\right) = f\left(\frac{\sigma_{cr}}{\sigma_e}\right) \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where f is a function of σ_{cr}/σ_e alone. Once the plate function f is established the compression flange width can be reduced continuously with increased applied moments through the relation represented by (5). For the converted section the corresponding beam properties A, B, C, C_w and β_1 can then be computed and are taken as the effective properties of the section. The procedure suggested for treating post-buckled beams therefore involves continuously transforming the original bisymmetrical I-beam into a monosymmetrical I and the original T-beam into a narrower T. In this manner the section is made locally stable at every stage of the loading and the fundamental equation, (4), developed for the critical moment of non-deformed sections can be used to evaluate the interacted critical load.

(b) *Selection of the Plate Function f*

The appropriate plate function can best be determined from the evidence of experimental tests. Guided initially by von Karman's general form the following expression for compression flange reduction was developed by trial from the test data reported in Part II :

$$\left(\frac{\bar{b}}{b}\right) = 0{\cdot}86\,\frac{t}{b}\sqrt{\frac{E}{\sigma_e}} \quad . \quad . \quad . \quad . \quad (6)$$

valid for $0{\cdot}48 \leq \frac{t}{b}\sqrt{\frac{E}{\sigma_e}} \leq 1{\cdot}16$, the range considered in the tests. E is the modulus of elasticity of the material and t the flange thickness. Stability calculations for sections transformed according to (6) provide a reasonable fit to the buckling strength obtained by tests for beams exhibiting an interaction of the instability modes.

Equation (6) indicates that local buckling commences $\left(\frac{\bar{b}}{b} = 1\right)$ at a stress of approximately two-thirds the theoretical critical for a flange having fixed-free boundaries along the unloaded edges. This is not particularly unusual; as a result of inevitable initial deviations from flatness plates normally buckle at stresses well below the critical and the results of (6) are therefore consistent with experimental observations, including those contained in Part II of this paper.

(c) *Method of Solution*

At any stage in the elastic post-buckling range the modified rigidities and the section property β_1 may be determined by standard calculations using a transformed section with effective compression flange width $\bar{b}$ corresponding to the junction stress σ_e as defined by (6). Denoting the values in the post-buckled state by bar quantities, the critical moment of a post-buckled beam may be written as :

$$M_{cr} = \frac{\pi^2 \bar{B}\bar{\beta}_1^2}{2\bar{\gamma}L^2} \pm \sqrt{\frac{\pi^4\bar{B}^2\bar{\beta}_1^2}{4\bar{\gamma}^2L^4} + \frac{\pi^2\bar{B}^2}{\bar{\gamma}L^2}\left(\bar{C} + \frac{4\pi^2\bar{C}_w}{L^2}\right)} \quad . \quad . \quad . \quad . \quad (7)$$

This must equal the moment of resistance of the converted beam by the conventional bending formula

$$M \quad \frac{\sigma_e}{\bar{c}}\left(\frac{A}{E}\right) \quad . \quad . \quad . \quad . \quad (8)$$

in which $\bar{c}$ is taken as the distance from the centroid of the section to the centre of the compression flange. For any selected value of σ_e, $\sigma_0 \leq \sigma_e \leq \sigma_y$, where σ_y corresponds to the 0·1 per cent proof stress and σ_0 defines the stress at which post-buckling reductions first become effective ($\bar{b}/b = 1$), the moments given by (7) and (8) are equated and the unknown critical span length, L_{cr}, is determined by solving the resulting quadratic in L^2, or simply by a process of trial and error.

For the elastic post-buckling range the $M-L$ curves shown in Part II have been evaluated in this manner.

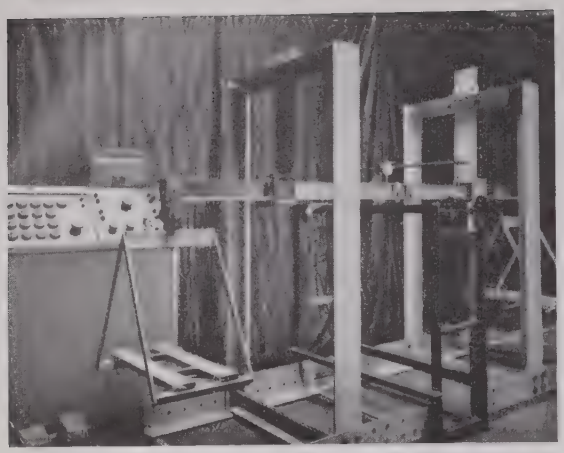

Fig. 3.—View of Typical Experimental Arrangement.

PART II

3. Experimental Programme

3.1 *Description of Tests, Apparatus, Specimens and Measurements*

To study the behaviour of beams failing in a combined 'local-cum-lateral' buckling mode six main series of tests were conducted in which specimen dimensions and material properties were varied. Five of the series, designated *P*, *A*, *B*, *C* and *D* were concerned with experiments on I-beams and the sixth or T-series with an investigation of T-beams. All specimens were loaded under conditions of pure end couples so that every element of beam length could be expected to contribute to the overall interaction effect to the same degree. The critical moment was obtained experimentally in each case and a record was made of onset of local flange buckling and of wave length. Various deflection and strain measurements were also taken on representative specimens of the different series.

A general view of the dead load system used for the tests is shown in Fig. 3. The apparatus consisted of two independent but identical end frames from which welded square boxes for loading and support were suspended by long thin steel plates. The box fittings were open at one end to receive the test specimens which were centered therein by appropriate packing pieces and securely clamped in position by means of set screws threaded into the walls of the boxes. This had the effect of rigidly preventing warping of the end sections of the beams. The extremely flexible plates acted as hinges and permitted the ends of the beams to rotate freely with respect to their principal axes while preventing twist about their longitudinal axes. At the same time, relative axial movement of the supports was possible thereby ensuring that no axial load developed. The use of plate-suspended box supports therefore fulfilled the idealized end conditions which lead to the critical state represented by equation (4). A uniform bending moment was applied to the beams by means of lever arms which were welded to the closed ends of the box fittings and which supported loading platforms carrying moulded lead weights. The general arrangement depicted allowed considerable flexibility in length and type of section which could be accommodated.

To provide a significant elastic range for the development of a locally post-buckled state of the outstanding compression elements it was necessary to reduce the flange thicknesses of various standard aluminium extrusions of I and T-form in a milling machine. The P-series specimens, which were used for preliminary tests to determine the feasibility of proceeding with more extensive experiments, were prepared in the department workshops and due to limitations imposed by the milling machine some variations in flange thickness were noted. However, for the principal test series *A*, *B*, *C*, *D* and T, extrusions of 14-foot lengths were modified in an aeronautical spar miller and a consistent set of specimens was obtained. Only the flanges were machined; the relatively thick webs were unaltered and did not experience any local buckling. The average measured cross section dimensions of the different series are summarized in Table I.

In series *A* and *T* the elastic range available for the development of the post-buckled state was relatively large and the conception of the instability load as the minimum moment at which the bifurcation condition could be established by a slight disturbing force provided a practical experimental method for evaluating the critical buckling moment. In the remaining series the available post-buckling range was comparatively small and generally bifurcation could not be established in this manner; any sideways disturbance caused immediate collapse. Consequently, the critical moment was taken as the moment causing collapse.

The deflection modes corresponding to local and lateral instability were established by independent means. A gauge based on the optical-lever principle was developed to detect the onset of local buckling and was arranged to measure the local deflection of the flange relative to the web. Gauges of this type were normally fitted in pairs to a test beam in the region of the half or quarter point of the span. An additional indication of the onset of plate buckling

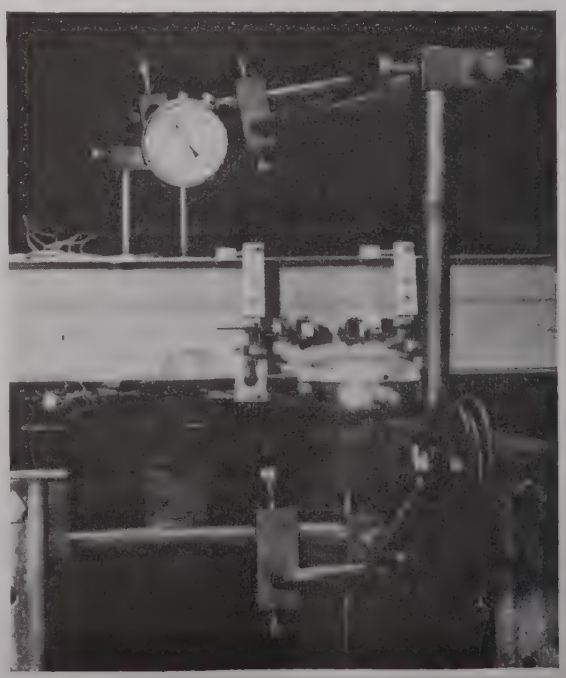

Fig. 4.—View of Instrumentation.

TABLE I

Average Section Dimensions and Mechanical Properties

Test Series	Overall depth ins.	Flange width ins.	Web thickness ins.	Flange thickness ins.	Material	0·1% Proof Stress t.s.i.	Modulus of Elasticity x10³ t.s.i.
P (I-beam)	1·863	1·504	0·109	0·0423	HE 15 WP	27·0	4·71
A ,,	2·845	3·015	0·145	0·0628	HE 15 WP	29·0	4·82
B ,,	2·915	3·014	0·149	0·0830	HE 11 WP	25·2	4·68
C ,,	2·842	3·005	0·202	0·0746	Noral 65 ST	17·2	4·60
D ,,	2·816	2·988	0·183	0·0749	HE 11 WP	24·3	4·80
T (T-beam)	2·451	2·518	0·185	0·0526	DTD 363 A	34·7	4·64

was obtained by the concept of strain reversal using pairs of electrical resistance strain gauges mounted along the unsupported edges on opposite faces of the flanges. At the bifurcation or collapse condition a trace of the local buckled mode was obtained and measurements of the wave length thereby recorded. Measurements of the horizontal and vertical deflection and rotation of the centre section of selected beams in each series were obtained by means of standard mechanical dial gauges. Fig. 4 shows the typical instrumentation arrangement used for the various series.

The mechanical properties of the alloy material from which the I and T-sections were extruded were determined from tensile tests on coupons removed from the web and machined flanges of each extrusion. The averages of the elastic values which were used in the stability calculations are summarized in Table I.

Standard bend and twist tests were performed on the machined extrusions in order to measure experimentally the average values of the flexural and torsional rigidities. These quantities were used in subsequent theoretical instability computations for the non-deformed sections.

TABLE II

Average Local Buckling Stress and Wave Length

Test	Local Buckling			Wave Length—ins.	
	Observed		Theory*		
	Moment in. tons	Flange Stress t.s.i.	Flange Stress t.s.i.	Measured	Theory**
P				2·04	2·35
A	4·97	7·28	9·70	4·97	4·83
B	10·50	11·30	16·48	4·88	4·82
C	9·36	10·63	13·22	4·27	4·72
D	9·00	10·53	13·98	4·37	4·72
T	3·02	7·95	9·43	3·80	3·92

* $\sigma_{cr} = \dfrac{K\pi^2 E}{12\,(1-\mu^2)}\left(\dfrac{t}{b/_2}\right)^2$ where $K = 1{\cdot}328$; $\mu = 0{\cdot}25$ (see Ref. 11).

** Wave length $= 2kw$ where $k = 1{\cdot}68$; w = clear flange width (see Ref. 12).

3.2 *Discussion of Results*

Results of the local buckling observations are summarized in Table II and compared with the theoretical values for flat plates based on fixed-free boundary conditions along the unloaded edges. The application of the principle of moment distribution[10] to the sections indicated that full restraint was provided at the web junction. In all cases the experimental critical stresses were lower than those derived from theoretical consideration, the order of magnitude being about 30 per cent. This is not excessive for experiments of this kind and can probably be attributed to the effects of imperfections and deviations from flatness. Average critical values have not been reported for the P-series due to inconsistencies resulting from the previously mentioned variations in flange thicknesses. Table II also compares the results of the average measured wave lengths with the corresponding theoretical values deduced from plate theory. The agreement is very reasonable. Although slight variations in wave lengths with location along the span were sometimes noted, the length of an individual wave remained essentially constant during a test.

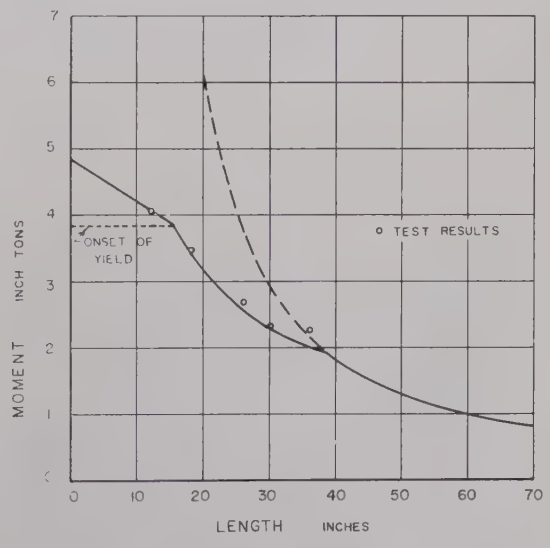

Fig. 5.—Comparison of Calculated and Experimental Results for P-Series I-Beams.

In Figs. 5 to 10 inclusive the observed critical moments for the beams of the different series are plotted against the span lengths and compared with the theory. The elastic buckling curves for the original non-distorted sections are shown on the figures as broken lines and are based on the average measured

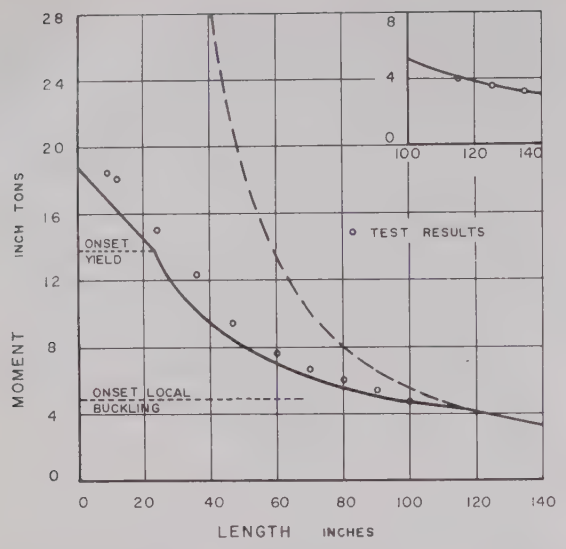

Fig. 6.—Comparison of Calculated and Experimental Results for A-Series I-Beams.

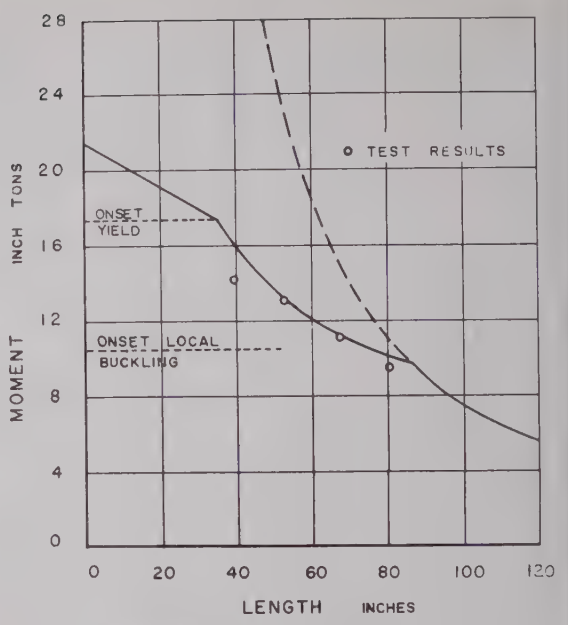

Fig. 7.—Comparison of Calculated and Experimental Results for B-Series I-Beams.

values of stiffnesses and elastic moduli. The post-buckling curves are shown as solid lines, the elastic portion being derived by the method set down in Part I. When local buckling and yielding progress simultaneously the plastic post-buckling variation is estimated by a straight line joining the critical moment for the transformed section corresponding to 0·1 per cent proof stress with the fully plastic moment for this same section. For this purpose the material was assumed as ideally elastic-plastic. It should be recognised that this simple treatment for the plastic branch of the curve is only very approximate. A detailed analysis for the conditions existing over this range is beyond the scope of this paper.

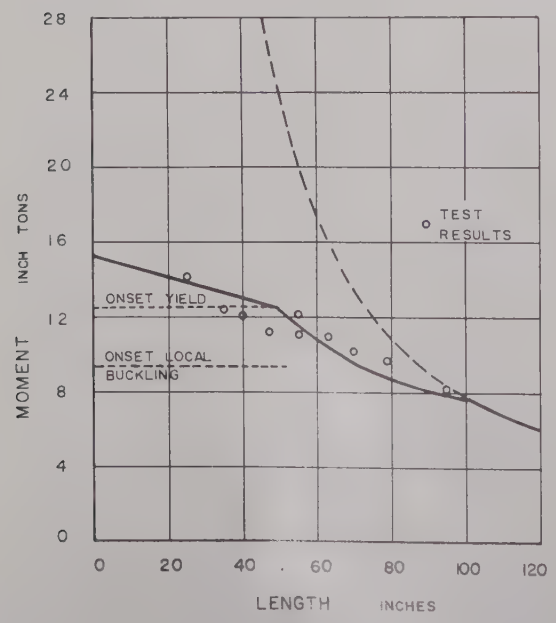

Fig. 8.—Comparison of Calculated and Experimental Results for C-Series I-Beams.

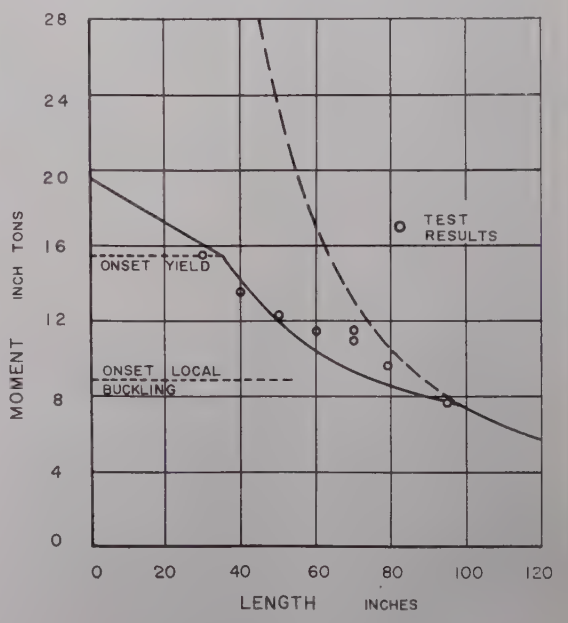

Fig 9—Comparison of Calculated and Experimental Results for D Series I-Beams.

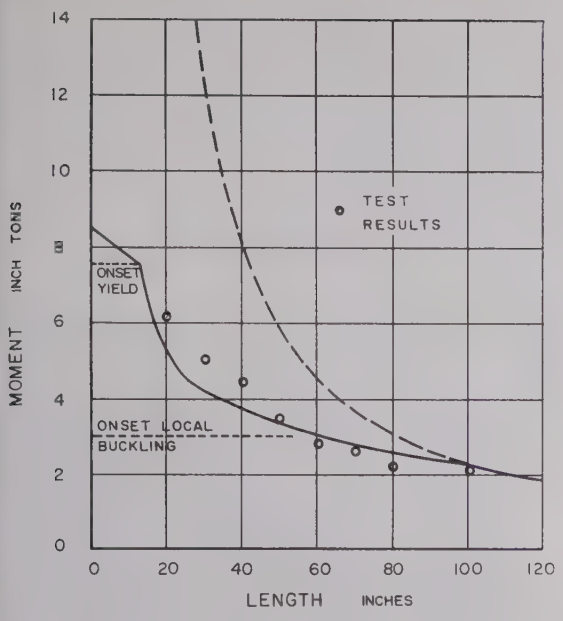

Fig. 10.—Comparison of Calculated and Experimental Results for T-Series T-Beams.

The post-buckling strength is markedly illustrated in the different figures. This strength decreases with increasing values of test span and tends towards the Euler curve as the critical length at which both local and overall instability are equal is approached. The experimental points form a discontinuous break from the Euler relation indicating the predominating influence of local buckling. In general, when the scatter of some of the results is explained, the transformed section solution can be seen to provide a reasonable estimate to the overall stability strength in the elastic post-buckling range.

The test results for the P-series beams are plotted on Fig. 5 from which it may be seen that a significant post-buckling strength was indicated. It was felt that these preliminary experiments were sufficiently promising to justify more detailed and extensive tests and it was on this basis that the five major series of the experimental programme were formulated.

The group *A* beams possessed a large elastic reserve for post-buckling development, the ratio of moment for developing a 0·1 per cent proof stress if local buckling was absent to moment causing plate instability, subsequently referred to as the reserve ratio, being 4. The experimental points, Fig. 6, exhibit a surprisingly small amount of scatter and may be seen to lie along a smooth curve. The test results and the theoretical equivalents for the three longest spans are reported separately in an insert in the upper portion of the figure. The measured stiffnesses for the extrusion from which these beams were taken were slightly below specification and were not included in the calculations for the averages of this series. These three beams did not suffer from noticeable local distortion and by their close agreement with theory tend to confirm a general trend indicated, namely, that the section is fully effective when local buckling is absent.

Because of their short spans, the 9 and 12-inch specimens were tested in a standard mechanical machine using a four point loading system. Load and support was applied through ball races to reduce lateral restraint and provide hinged conditions. Failure was due to a combination of lateral movement of the compression flange and tearing of the material at the web-flange junction, presumably due to the large local distortions.

The beams of the B-series had a reserve ratio of 2·2 and were designed so that the experimental points, Fig. 7, might exhibit a more marked discontinuity at their point of departure from the Euler curve than that noticed with the A-group. Due to the limited number of tests involved this series was not permitted to influence strongly the empirical relation ultimately adopted for section transformation. Nevertheless, the suggested approach still provided a reasonable estimate to the interacted instability strength.

The *C* and *D* tests, Figs. 8 and 9, were undertaken to study the effect of the relative occurrence of onset of yield and local instability on the buckling strength. Both groups were designed to have identical section dimensions but different 0·1 per cent proof limits. The D-specimens were selected to exhibit an elastic post-buckling condition and the reserve ratio actually obtained with this group was 2·3. The C-series was designed so that local buckling and yielding commenced simultaneously but due to deviations from the expected proof and buckling stresses an unintended reserve ratio of 1·6 was experienced by the members of this group. Unlike the behaviour of thin-walled columns, due to the presence of an elastic core, plastic buckling of beam flanges need not lead to immediate collapse of the parent section. However, for small elastic reserve ratios, under the combined action of local deformation and plastic growth this core would be rapidly depleted and the instability load would increase relatively little over a range of test spans.

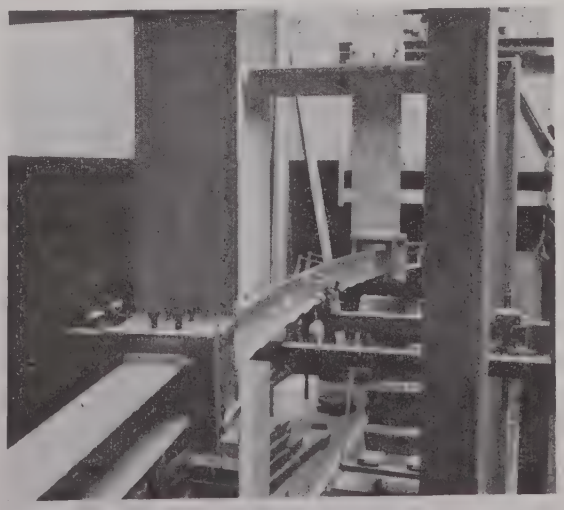

Fig. 11.—50-inch T-Series Beam at Bifurcation Condition.

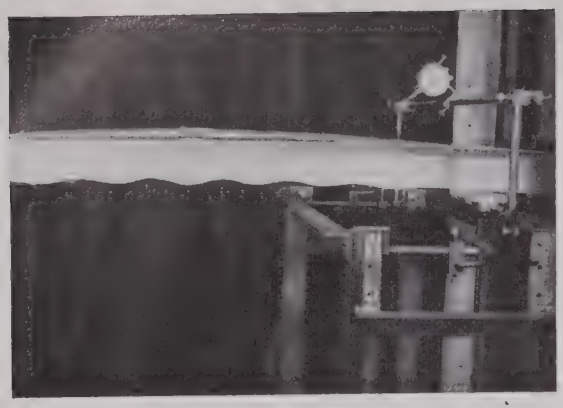

Fig. 12.—48-inch A-Series Beam at Collapse Condition.

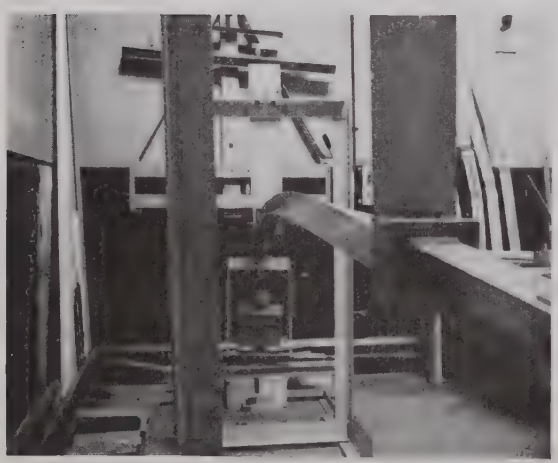

Fig. 13.—Two Views of 95-inch C-Series Beam at Collapse Condition.

While there is a tendency in this direction for the C-beams, due to the limited number of tests involved and the degree of scatter observed firm conclusions on this point are not justified. The scatter noted in the C-results can be attributed, at least in part, to variations in material properties; the 0·1 per cent proof stresses and the elastic moduli of the two different extrusions from which the 55-inch specimens were removed differed by 17 per cent and 6 per cent respectively. As expected, the group D-beams were capable of supporting the greatest loads.

Fig. 14.—Typical Local Wave Shapes. Centre Section of 53-inch B-Series Beam at Collapse Condition.

The application of the transformation principle to sections originally monosymmetric is illustrated in Fig. 10, which presents the results obtained with beams of T-form. The specimens of this series exhibited a reserve ratio of 4·3 and behaved in a manner similar to that observed with the group A sections. The rigidities of the T-beams were relatively small and in the longer spans sizeable lateral deflections and rotations were recorded from the beginning of loading. It is probable that this contributed to the strength reductions noted with the longer spans.

Figs. 11, 12 and 13 present respective views of the 50-inch T-beam at bifurcation, the 48-inch A-beam at collapse and the 95-inch C-beam at collapse. A close up of the wave shapes at the centre section of the 53-inch B-beam at the collapse condition is shown in Fig. 14.

3.3 *Concluding Remarks*

The experiments reveal that the phenomenon of interaction between local and lateral buckling can be a very real and significant feature of the general instability problem. The margin of strength exhibited by beams beyond the first occurrence of local instability may be appreciable and varies widely according to the span length and material of the section. This strength decreases with increasing span since beam buckling then becomes increasingly important and predominates. When material yielding and local failure proceed simultaneously indications are that the post-buckling strength is rapidly exhausted and collapse takes place suddenly. The agreement obtained between the calculated and observed results supports the suggestion that the reduction in strength of post-buckled beams may be accounted for through the concept of a transformed section as outlined in the main body of this paper.

Acknowledgements

The work reported in this paper was carried out in the Department of Civil Engineering, University of Bristol. The writer is especially grateful to Professor Sir Alfred Pugsley for his continuous encouragement and advice during the course of these studies.

The support of the British Government and the National Research Council of Canada, who provided grants during the period of this research, is gratefully acknowledged. Special mention is due to the Bristol Aeroplane Company, who presented the University with a large number of the specimens used in the experimental investigations.

Appendix

The Section Property β_1

Since the term β_1 is a function of $\bar{b}$ it must also be modified continuously in post-buckled beams. It may be shown[2] that the expression for β_1 is given by

$$\beta_1 = \frac{\int_{A'} y^3 dA' \quad \int_{A'} x^2 y dA'}{I_x} + \quad - 2y_o$$

where x, y define the coordinates of any point in the section measured relative to the centroidal principal axes and y_o is the coordinate of the shear centre with respect to the same origin. I_x is the major centroidal moment of inertia of the section. Upon integrating over the section area A' and simplifying a systematic computation schedule may be set down for β_1.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by November 30th, 1960.

Book Review

Fracture. Proceedings of an International Conference on the Atomic Mechanisms of Fracture, held in Swampscott, Massachusetts, April 12-16, 1959. Edited by B. L. Averbach, D. K. Felbeck, G. T. Hahn and D. A. Thomas. (The Technology Press of Massachusetts Institute of Technology, and New York : John Wiley, London : Chapman & Hall, 1959). 9 in. × 6 in., 646 + xvii pp. 140s.

This conference, sponsored by the National Science Foundation Office of Naval Research, the Air Force Office of Scientific Research and the Ship Structure Committee and organized and directed by the Conference on Fracture of the National Academy of Sciences National Research Council, was attended by experts in the field from many countries.

The Conference was concerned with the question of what actually occurs on an atomic scale when materials fracture, and the papers dealt with current theories and experiments on cleavage, ductile, fatigue and high-temperature mechanisms of fracture for metals, ceramics and polymers, but the field of engineering fracture mechanics was not included.

During the first part of the Conference the twenty-seven papers printed in this volume were delivered at session open to the scientific public. After this, invited members met as a Fracture Research Planning Conference to consider the current status of the theoretical and experimental problems that had been presented during the public sessions. The summarizing reports of the chairmen of the Conference meetings constitute the first chapter of this volume.

The book is extremely well produced and illustrated and each paper contains a bibliography of related references.

Groups of Piles Under Mono-Planar Forces

by L. D. Turzynski, A.M.I.Struct.E.

INTRODUCTION

To transfer loads of structures through soft upper layers of soil to harder strata below, piling is currently adopted.

Considerable literature has therefore appeared, in which attempts are made to obtain means of solving the statically indeterminate pile groups by making reasonable simplifying assumptions and adopting known methods of analysis to arrive at the actual loads in the piles.

Most of the available methods, however, lack speed and flexibility, and the effects of the various steps in the analysis are not clearly illustrated, mainly because the approach is not essentially a physical one.

The general solution usually given by a series of equations expressed in general terms, is very laborious, time consuming, prone to errors and does not give the designer the ' feel ' of the structure.

The semigraphical method of approach, although fairly straightforward to use is very time consuming especially if one considers a multipile asymmetrical pile cap subjected to several load conditions.

In the relaxation or indirect method of approach in which the out of balance forces and moments are gradually eliminated by an iterative process, the convergence may be so slow in practice, that the necessary amount of work may become prohibitive. Furthermore, no advantage can be taken of one solution for a given pile cap for checking the effects of several other load conditions if present.

As in the majority of cases several load conditions must be considered, for example in bridge abutments, the disadvantages of the above methods are obvious. A method most suitable from this point of view would be one in which the applied loads could be separated entirely from the geometrical and physical properties of the pile cap, so that any alteration in the applied loads would not affect that part of the solution which is governed by the physical and geometrical properties only.

The method of influence coefficients has suggested to the author a possible approach which would meet most of the requirements stated above. The analysis is divided into simple steps of purely numerical calculations which can be readily checked. By tabulating the various steps the operations can be ' streamlined ' for an efficient office procedure. The final result gives a set of equations which express the loads in the piles in terms of the applied loads, and hence any combination of them can easily be checked.

ASSUMPTIONS

The general assumptions upon which a solution is obtained for a group of piles acted upon by forces and a moment in one plane only are listed below :—

1. Passive resistance of soil to horizontal forces is negligible.
2. Piles are assumed hinged at the head and toe.
3. The rigidity of the pile cap is such that the effect of its deformation is negligible compared with the deformation of the individual piles.
4. Within the range of permissible loading all piles deform elastically.
5. Young's modulus, cross-sectional area and length for individual piles are constant.

Since there is already an extensive literature on the views of engineers regarding assumptions 1 to 4 they will not be discussed here, and reference should be made to the bibliography listed at the end of this article (1) (2) (3) and (4).

Assumption 5 is not essential for obtaining the solution but since it is very unlikely that for any given foundation more than one size of pile would be adopted especially if bored reinforced concrete piles are used, then the assumption seems to be justified, considering that it somewhat simplifies the arithmetic. Nevertheless, should such a case arise the relative stiffnesses of the various piles can easily be evaluated and included in the solution (5).

THEORY

1. General Considerations—

Under the action of external forces and a moment the assumed rigid pile cap will be translated to a definite final position in the plane considered. Any position in that plane can in fact be reached by three basic displacements i.e. vertical, horizontal and rotational here denoted by Δ_V, Δ_H and Δ_R respectively.

To evaluate the final load on any pile in a group, the three basic displacements must first be determined. This can be done very conveniently by adopting the method of influence coefficients, which can be applied to many problems of statically indeterminate structures (6). The influence coefficients are defined as the effects of unit displacements of the pile cap on the individual piles, and are derived in Part 2 below.

Using matrix notation, the influence coefficients for a given pile can be put in the form of a flexibility matrix which will express the sensitivity of that pile to the three unit displacements. The sum of the flexibility matrices for the individual piles is then equal to the flexibility matrix for the pile group. The method of evaluating the final displacements using the flexibility matrix for the group will be described in Part 2.

The derivation of the influence coefficients or the elements of the flexibility matrix for a pile inclined at an angle $\varnothing$ to the vertical and distant x from the centre of rotation will be carried out separately for

the three basic unit displacements and the result summed up.

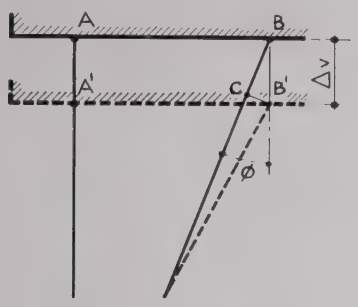


FIG. 1.

2. Derivation of the elements of the Flexibility Matrix—

(a) *Unit vertical displacement* $\Delta v = 1$.

If the pile cap shown on Fig. 1 deflects vertically downwards by $\Delta v = 1$ then the deformation of a vertical pile is equal to $AA' = 1$ and the deformation of a raking pile is equal to

$$BC = BB' \cos \varnothing = \cos \varnothing$$

If the reaction on the vertical pile caused by the unit deflection of the cap is assumed to be unity, then the axial reaction on a raking pile will be equal to $\cos \varnothing$

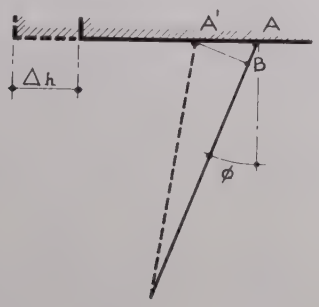


FIG. 2.

(b) *Unit horizontal displacement* $\Delta h = 1$.

When the pile cap deflects horizontally $\Delta h = 1$ (see Fig. 2) there is no axial deflection for a vertical pile (the slight increase in length considered negligible) and therefore no reaction. The deflection of a raking pile is equal to

$$AB = AA' \sin \varnothing = \sin \varnothing$$

and, as before, the axial reaction on a raking pile is equal to $\sin \varnothing$.

(c) *Unit rotation* $\Delta r = 1$.

Due to unit rotation $\Delta r = 1$ (see Fig. 3) the head of the pile will deflect from A to A' where

$$AA' = x\Delta r = x$$

Therefore the deflection of any raking pile is

$$AB = AA' \cos \varnothing = x \cos \varnothing$$

consequently, as above, the axial reaction on a raking pile due to unit rotation is equal to $x \cos \varnothing$.

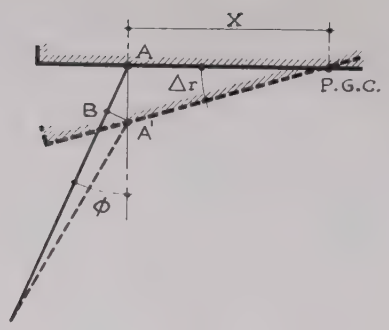


FIG. 3.

Summarizing therefore, the total axial reaction on any raking pile due to the three displacements is given by :—

$$\cos \varnothing + \sin \varnothing + x \cos \varnothing$$

Since the externally applied loads acting on the pile cap can be conveniently expressed in terms of the three resultants V, H and M, i.e. a vertical and horizontal force and a moment about any point, taken here as the pile group centre, as defined later, then the effects of the axial reactions of the individual piles on the pile cap due to the unit displacements will have to be expressed in similar forces and a moment. If we denote the vertical and horizontal components of the axial force and the moment that the vertical component will produce at the pile group centre by v, h and m respectively, then using matrix notation they are given by :—

$$\begin{bmatrix} v \\ h \\ m \end{bmatrix} = \begin{bmatrix} \cos \varnothing \\ \sin \varnothing \\ x \cos \varnothing \end{bmatrix} \begin{bmatrix} \cos \varnothing & 0 & 0 \\ 0 & \sin \varnothing & 0 \\ 0 & 0 & x \cos \varnothing \end{bmatrix}$$

carrying out the row by column multiplication

$$\begin{bmatrix} v \\ h \\ m \end{bmatrix} = \begin{bmatrix} \cos^2 \varnothing & \cos \varnothing \sin \varnothing & x \cos^2 \varnothing \\ \cos \varnothing \sin \varnothing & \sin^2 \varnothing & x \cos \varnothing \sin \varnothing \\ x \cos^2 \varnothing & x \cos \varnothing \sin \varnothing & x^2 \cos^2 \varnothing \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad (1)$$

Since the forces v and h and the moment m are the results of unit deformations the matrix on the right hand side of the equation is the flexibility matrix and its elements the flexibility coefficients for any given raking pile. Here it must be remembered that for a vertical pile the angle $\varnothing = 0$ and therefore $\cos \varnothing = 1$ and $\sin \varnothing = 0$ which on substitution give the flexibility matrix for a vertical pile.

As one would expect, the flexibility matrix is a symmetrical one since for example the vertical reaction due to a horizontal deflection must be equal to the horizontal reaction due to a vertical deflection etc.

The flexibility matrix for the pile group as stated before is equal to the sum of the flexibility matrices for the individual piles. Therefore, in general terms the flexibility matrix for the whole group takes the form :—

$$\begin{bmatrix} a_{11} & a_{12} & a_{13} \\ a_{21} & a_{22} & a_{23} \\ a_{31} & a_{32} & a_{33} \end{bmatrix} \quad \cdot \quad \cdot \quad (2)$$

where each element is equal to the sum of the elements of the individual pile flexibility matrices, or the appropriate individual pile influence coefficients.

i.e. $a_{11} = \Sigma \cos^2 \varnothing \quad a_{12} = \Sigma \cos \varnothing \sin \varnothing$ etc.

The relationship between the applied forces and moments about the pile group centre and the three final basic displacements is then given by :

$$\begin{bmatrix} V \\ H \\ M \end{bmatrix} = \begin{bmatrix} \Delta_V \\ \Delta_H \\ \Delta_R \end{bmatrix} \begin{bmatrix} a_{11} & a_{12} & a_{13} \\ a_{21} & a_{22} & a_{23} \\ a_{31} & a_{32} & a_{33} \end{bmatrix} \quad . \quad . \quad (3)$$

If there is only one load condition acting on the pile cap then direct solution of the three simultaneous equations will yield the required total displacements Δ_V, Δ_H and Δ_R.

If however, as for instance on bridge abutments there are several loading conditions to be analysed, then the procedure would be as follows :

Let the load matrix be represented by P, the deflection matrix by Δ and the flexibility matrix by A, then the relationship (3) can be written down as

$$P = \Delta A$$

Since at this stage we are interested in the deflections, the above relationship can be transformed to

$$\Delta = PA^{-1}$$

where A^{-1} is the inverse of A (7)

This will enable us to express the three basic deflections in terms of the applied loads and moment.

3. Loads on Piles—

Having determined the total displacements due to the applied loads either by direct solution or by inversion of the matrix A, the vertical and horizontal reactions and the moment that the pile will exert on the pile cap at the chosen pile group centre i.e. F_V, F_H and F_R respectively acting on any individual raking pile are then given by the product of the final displacements and the flexibility matrix for that pile. In the case of a vertical pile, the appropriate flexibility matrix should of course be used.

$$\begin{bmatrix} F_V \\ F_H \\ F_R \end{bmatrix} = \begin{bmatrix} \Delta_V \\ \Delta_H \\ \Delta_R \end{bmatrix} \begin{bmatrix} \cos^2\varnothing & \cos\varnothing \sin\varnothing & x\cos^2\varnothing \\ \cos\varnothing \sin\varnothing & \sin^2\varnothing & x\cos\varnothing \sin\varnothing \\ x\cos^2\varnothing & x\cos\varnothing \sin\varnothing & x^2\cos^2\varnothing \end{bmatrix}$$

. . . (4)

Since, as can be seen $F_H = F_V \tan\varnothing$ and $F_R = F_V x$ only the vertical reaction or load on the pile need be evaluated using the first equation from (4). The horizontal force and the moment will be evaluated, when checking the statical equilibrium of the group.

The vertical component of the load on a raking pile is therefore from (4)

$$F_V = \Delta_V \cos^2\varnothing + \Delta_H \cos\varnothing \sin\varnothing + \Delta_R x \cos^2\varnothing \quad (5)$$

for a vertical pile this will reduce to

$$F_V = \Delta_V + x\,\Delta_R \quad . \quad . \quad . \quad . \quad . \quad (6)$$

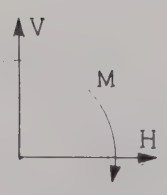


FIG. 4.

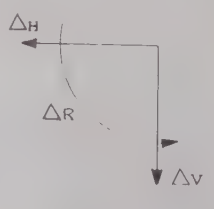


FIG. 5.

SIGN CONVENTION

As in any engineering structure a certain sign convention has to be adopted. The one chosen by the author is given in Fig. 4 and Fig. 5. Fig 4 gives the sign convention for the externally applied forces and moment, and Fig. 5 for the three basic displacements. The directions shown are assumed to be positive.

Any sign convention can be adopted, but not all will give a symmetrical flexibility matrix as far as the signs of the various elements are concerned, which would somewhat complicate the arithmetic when inversing the matrix A.

PILE GROUP CENTRE

When evaluating the shortening of piles due to unit rotation, and also the final loads on piles, reference has been made to the pile group centre.

Generally speaking this can be chosen at any point in the plane considered, and a solution will be obtained. However, to simplify the calculations and to reduce the sensitivity of the simultaneous equations it is advisable to make as many elements of the flexibility matrix as possible equal to zero.

When evaluating the shortening of the piles due to vertical displacement, one can see that if the effective 'area' of a vertical pile is taken as unity, that of a raking pile will be only $\cos^2\varnothing$. Hence by taking moments about a suitable point of the effective areas, the centre of gravity of those 'areas' can be determined. When the forces, moment, displacements and pile positions are related to this point which we shall call pile group centre, the third element of the first row and column of the flexibility matrix becomes zero for an asymmetrical arrangement of piles and the second and third elements of the first row and column become zero for a symmetrical arrangement of piles. The advantages in this case are obvious. It should be noted that the pile group centre chosen is not the conventional pile group centre as defined by other writers.

GENERAL REMARKS

The theoretical treatment of the pile group analysis may seem fairly long and elaborate, but in fact the various calculations are very straightforward as the examples that follow will illustrate.

The first example given is a symmetrical group of piles subjected to one loading condition, in which the horizontal force H and the moment can act in both directions. This example has been worked out in more detail to show clearly the various steps in the analysis.

The second example has an asymmetrical arrangement of piles. The loads on piles have been expressed in general terms i.e. by the use of the inversion of the flexibility matrix,—a procedure useful to adopt with several loading conditions. The calculations in this case are presented in a more abbreviated form, but nevertheless should be easy to follow.

Example No. 3 illustrates the method to be adopted, when the longitudinal spacing of the individual piles in the group differs. Here the flexibility matrix is evaluated for a unit group, which repeats in the longitudinal direction of the wall. This is equivalent to making the stiffness of the substitute pile in any row inversely proportional to the longitudinal spacing of the actual piles. The remaining calculations follow as in Example No. 1 or 2.

Matrix notation has been adopted in this article for brevity. Those unfamiliar with matrix technique should note that the treatment adopted requires only the solution of three simultaneous equations during analysis.

Due to the large number of variables i.e. the number of piles, their rake and spacing, the usual procedure adopted in the design of pile caps is a 'guess' by the designer and subsequent analysis,—the speed with which a suitable pile cap is found depending entirely on the experience of the designer.

The procedure could be improved considerably if use were made of an electronic digital computer, as the method presented in this article would be ideal for that purpose. When writing a suitable programme, the rakes of the piles, their spacing or both could be made variable in a predetermined order, and a conditional jump instruction inserted,—the condition being the limiting maximum and minimum load on any pile. If on completing the first analysis, the conditions given were not satisfied, the computer would ' jump ' back to the beginning of the programme, select revised rake of pile or its spacing and repeat the analysis, thus forming a repetitive loop, until the given conditions were satisfied. The number of repetitive loop instructions carried out by the computer is immaterial if one takes into account the speed with which the computer would perform the analysis.

The programme would in fact be an ' optimizing ' programme and current studies of such a programme are in progress. A severe difficulty, however, lies in the choice of criteria. For one load condition, such a choice would be simple and a narrow range of criteria could be established. Due to the many load conditions encountered in e.g. bridge abutments even one optimum pile layout may result in some piles being loaded with very little load for a particular load condition. Consequently the range given in the criteria must increase which in turn may result in a computer answer which is not optimum. This problem should not prove insoluble.

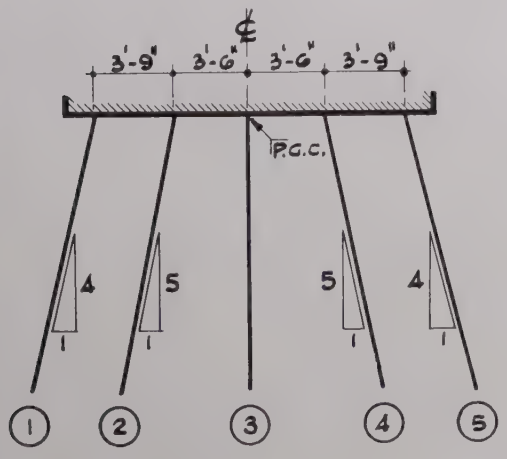


FIG. 6.

EXAMPLE No. 1

The group of piles shown on Fig. 6 is subjected to a vertical load of — 425·60 kips a horizontal force of — 22·40 kips and a moment of — 487·42 k.ft. acting about the ℄ of the pile cap. The forces and moment given have been evaluated for a 3 ft. 6 in. length of wall which is the assumed longitudinal spacing of piles. Piles 1 and 5 rake 1 in 4, piles 2 and 4 rake 1 in 5 and pile 3 is vertical. The trigonometrical constants for the given rakes are given in Table 1.

By inspection the pile group centre (P.G.C.) will be at the head of pile No. 3.

Functions of the angle of rake ø	For Rake 1 in 4	For Rake 1 in 5
tan ø	0·2500	0·2000
cos ø	0·9704	0·9805
sin ø	0·2425	0·1961
cos^2 ø	0·9413	0·9614
sin^2 ø	0·0588	0·0385
cos ø *sin* ø	0·2353	0·1923

TABLE 1.

The evaluation of the elements of the ' flexibility matrix ' for the group is given in Table 2. Each column of the matrix is evaluated separately as shown. The sketch above the column shows the directions and values in general terms of the loads induced in the piles due to the three basic deflections. All the information required for the evaluation of the column of the matrix is given on the sketch above it. The first element of each column is equal to the sum of the vertical reactions of all piles, the second element to the sum of the horizontal reactions of all piles, and the third element to the sum of the moments of all piles about the P.G.C. The signs of the respective forces must of course conform to the sign convention adopted.

The deflections are therefore given by :

$$-\begin{bmatrix} V \\ H \\ M \end{bmatrix} = \begin{bmatrix} \Delta_V \\ \Delta_H \\ \Delta_R \end{bmatrix} \cdot \begin{bmatrix} 4\cdot8054 & 0 & 0 \\ 0 & 0\cdot1946 & 4\cdot3304 \\ 0 & 4\cdot3304 & 103\cdot0816 \end{bmatrix}$$

Substituting $V = -425\cdot60$ kips $H = -22\cdot40$ kips and $M = -487\cdot42$ k. ft. and solving, the deflections are : $\Delta_V = +88\cdot5671$, $\Delta_H = +151\cdot6520$, $\Delta_R = -1\cdot6424$ The vertical loads on piles are given by

$$F_1 = +0\cdot9413\ \Delta_V + 0\cdot2353\ \Delta_H + 6\cdot3538\ \Delta_R$$

$$F_2 = +0\cdot9614\ \Delta_V + 0\cdot1923\ \Delta_H + 2\cdot8842\ \Delta_R$$

$$F_3 = +\ \Delta_V$$

$$F_4 = +0\cdot9614\ \Delta_V - 0\cdot1923\ \Delta_H - 2\cdot8842\ \Delta_R$$

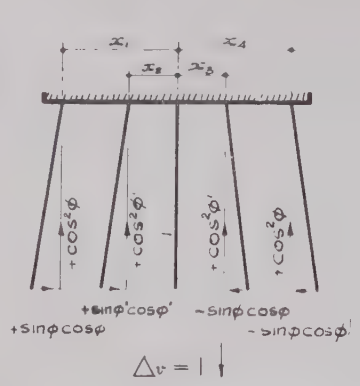
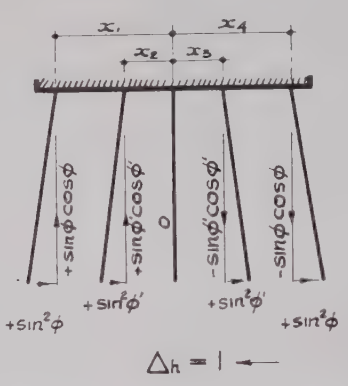
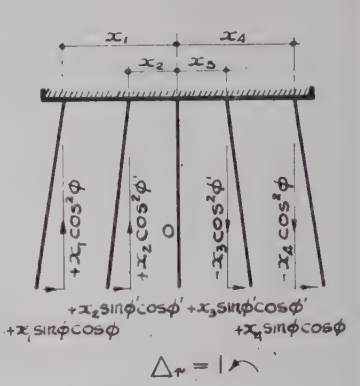
$$F_5 = +0\cdot9413\ \Delta_V - 0\cdot2353\ \Delta_H - 6\cdot3538\ \Delta_R$$

Substituting for Δ_V, Δ_H and Δ_R, the vertical and axial loads on the piles are then evaluated and the statical equilibrium checked in Table 3. When the force H and moment M act in opposite direction, piles 5 and 4 will be the most heavily loaded ones, while pile No. 3 will have the same load on it.

The last row of Table 3 gives the total vertical and horizontal forces and the total moment that the piles exert on the pile cap. These as stated before must be equal to the applied forces but with opposite signs.

EXAMPLE No. 2

Fig. 7 shows the arrangement of piles under the base slab of a bridge abutment. The loads per foot length on the abutment are given in Table 4 in which the moments have already been related to the pile group centre. To illustrate the procedure, although only four load combinations are given, a general solution will be obtained using the inversion of the ' flexibility matrix.' For trigonometrical constants see Table 1.

FIRST COLUMN OF MATRIX	SECOND COLUMN OF MATRIX	THIRD COLUMN OF MATRIX
		

Pile No.	a_{11}	a_{21}	a_{31}	a_{12}	a_{22}	a_{32}	a_{13}	a_{23}	a_{33}
1	0·9413	0·2353	6·3538	0·2353	0·0588	1·5883	6·3538	1·5883	42·8882
2	0·9614	0·1923	3·8842	0·1923	0·0385	0·5769	2·8842	0·5769	8·6526
3	1·0000	0·0000	0·0000	0·0000	0·0000	0·0000	0·0000	0·0000	0·0000
4	0·9614	— 0·1923	— 2·8842	— 0·1923	0·0385	0·5769	— 2·8842	0·5769	8·6526
5	0·9413	— 0·2353	— 6·3538	— 0·2353	0·0588	1·5883	— 6·3538	1·5883	42·8882
Total Σ	4·8054	0·0000	0·0000	0·0000	0·1946	4·3304	0·0000	4·3304	103·0816

TABLE 2.

Pile No.	Vertical Loads F (kips)	Axial Loads on Piles $F/\cos\phi$ kips	Axial Loads on Piles $F/\cos\phi$ tons	Horizontal Forces $F\tan\phi$ (kips)	Moments About P.G.C. Fx k. ft.
1	108·62	111·93	49·97	27·15	733·17
2	109·58	111·75	49·89	21·91	328·72
3	88·57	88·57	39·54		
4	60·73	61·94	27·65	— 12·14	— 182·16
5	58·10	59·90	26·74	— 14·52	— 392·31
Total	425·60			22·40	487·42

TABLE 3.

Loading Combination	Vertical Load V (kips)	Horizontal Load H (kips)	Moment About P.G.C. M (kip. ft.)
I	— 57·89	— 11·06	— 127·63
II	— 44·90	— 4·50	— 71·04
III	— 51·77	+ 2·50	+ 124·78
IV	— 90·40	— 5·15	+ 41·34

TABLE 4.

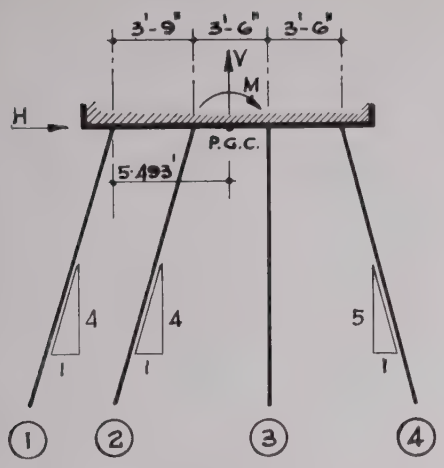


FIG. 7

Equivalent ' area '

$= 2 \times 0{\cdot}9413 + 1 + 0{\cdot}9614 = 3{\cdot}8440$

Pile group centre

$$= \frac{0{\cdot}9413 \times 3{\cdot}75 + 7{\cdot}25 + 0{\cdot}9614 \times 10{\cdot}75}{3{\cdot}8440}$$

$= 5{\cdot}4930$ from pile No. 1.

The elements of the ' flexibility matrix ' are as follows :

$a_{11} = 1 + 2 \times 0{\cdot}9413 + 0{\cdot}9614 = 3{\cdot}8440$

$a_{12} = a_{21} = 2 \times 0{\cdot}2353 - 0{\cdot}1923 = 0{\cdot}2783$

$a_{13} = a_{31} = 0$

$a_{22} = 2 \times 0{\cdot}0588 + 0{\cdot}0385 = 0{\cdot}1561$

$a_{23} = a_{32} = 0{\cdot}2353 \times (5{\cdot}493 + 1{\cdot}743) + 0{\cdot}1923 \times (5{\cdot}257) = 2.7135$

$a_{33} = 0{\cdot}9413 \times (5{\cdot}493^2 + 1{\cdot}743^2) + 1{\cdot}757^2 + 0{\cdot}9614 \times (5{\cdot}257)^2 = 60{\cdot}9182$

Therefore :

$$-\begin{bmatrix} V \\ H \\ M \end{bmatrix} = \begin{bmatrix} \Delta V \\ \Delta H \\ \Delta R \end{bmatrix} \begin{bmatrix} 3{\cdot}8440 & 0{\cdot}2783 & \cdot \\ 0{\cdot}2783 & 0{\cdot}1561 & 2{\cdot}7135 \\ \cdot & 2{\cdot}7135 & 60{\cdot}9182 \end{bmatrix}$$

Inversion of matrix using Foxe's Method (7).

$$A \begin{bmatrix} 3{\cdot}8440 & 0{\cdot}2783 & \cdot \\ 0{\cdot}2783 & 0{\cdot}1561 & 2{\cdot}7135 \\ \cdot & 2{\cdot}7135 & 60{\cdot}9182 \end{bmatrix} \quad \begin{matrix} \Sigma\, 4{\cdot}1223 \\ 3{\cdot}1479 \\ 63{\cdot}6317 \end{matrix}$$

$$L \begin{bmatrix} 1{\cdot}9606 & \cdot & \cdot \\ 0{\cdot}1419 & 0{\cdot}3686 & \cdot \\ \cdot & 7{\cdot}3607 & 2{\cdot}5958 \end{bmatrix}$$

S_1 $2{\cdot}1025$ $7{\cdot}7293$ $2{\cdot}5958$

$$(L^{-1})^1 \begin{bmatrix} 0{\cdot}5100 & -0{\cdot}1964 & 0{\cdot}5569 \\ \cdot & 2{\cdot}7126 & -7{\cdot}6924 \\ \cdot & \cdot & 0{\cdot}3852 \end{bmatrix}$$

S_2 $0{\cdot}5100$ $2{\cdot}5162$ $-6{\cdot}7503$

$$A \begin{bmatrix} 0{\cdot}6088 & -4{\cdot}8164 & 0{\cdot}2145 \\ -4{\cdot}8164 & 66{\cdot}5259 & 2{\cdot}9633 \\ 0{\cdot}2145 & -2{\cdot}9633 & 0{\cdot}1484 \end{bmatrix}$$

S_3 $-3{\cdot}9931$ $58{\cdot}7462$ $-2{\cdot}6004$

Therefore

$$\begin{bmatrix} \Delta V \\ \Delta H \\ \Delta R \end{bmatrix} = -\begin{bmatrix} V \\ H \\ M \end{bmatrix} \begin{bmatrix} 0{\cdot}6088 & -4{\cdot}8164 & 0{\cdot}2145 \\ -4{\cdot}8164 & 66{\cdot}5259 & -2{\cdot}9633 \\ 0{\cdot}2145 & -2{\cdot}9633 & 0{\cdot}1484 \end{bmatrix}$$

From which

$\Delta V = -0{\cdot}6088\, V + 4{\cdot}8164\, H - 0{\cdot}2145\, M$

$\Delta H = +4{\cdot}8164\, V - 66{\cdot}5259\, H + 2{\cdot}9633\, M$

$\Delta R = -0{\cdot}2145\, V + 2{\cdot}9633\, H - 0{\cdot}1484\, M$

The vertical loads on piles are given by

$F_1 = 0{\cdot}9413\, \Delta V + 0{\cdot}2353\, \Delta H + 5{\cdot}1706\, \Delta R$

$F_2 = 0{\cdot}9413\, \Delta V + 0{\cdot}2353\, \Delta H + 1{\cdot}6407\, \Delta R$

$F_3 = \Delta V - 1{\cdot}7570\, \Delta R$

$F_4 = 0{\cdot}9614\, \Delta V - 0{\cdot}1923\, \Delta H - 5{\cdot}0541\, \Delta R$

Substituting for ΔV, ΔH and ΔR.

$F_1 = -0{\cdot}5489\, V + 4{\cdot}2022\, H - 0{\cdot}2719\, M$

$F_2 = +0{\cdot}2083\, V - 6{\cdot}2579\, H + 0{\cdot}2519\, M$

$F_3 = -0{\cdot}2319\, V - 0{\cdot}3901\, H + 0{\cdot}0462\, M$

$F_4 = -0{\cdot}4274\, V + 2{\cdot}4466\, H - 0{\cdot}0260\, M$

A check will be made at this stage, by applying $V = H = M = -1$ to the pile cap. The individual vertical and horizontal forces and the moment that the piles exert on the pile cap are given in Table 5. The summation of the forces and moments should of course be equal to the applied forces but with a different sign.

Pile No.	Vertical Force Fv	Horizontal Force $F_H = Fv \tan \phi$	Moment About P.G.C. $F_R = Fv x$
1	−3·3814	−0·8453	−18·5740
2	5·7977	1·4494	10·1054
3	0·5758		−1·0117
4	−1·9932	0·3986	10·4782
Total	0·9989	1·0027	0·9979

TABLE 5.

The evaluation of the pile loads due to the four load combinations is given in Table 6.

The maximum load occurs on pile No. 1. If the maximum permissible load on the pile is 50 tons, then the longitudinal spacing of the piles will be :

$$\frac{50}{11{\cdot}5248} = 4{\cdot}33 \text{ say } 4' - 3'' \text{ centres}$$

EXAMPLE No. 3

The plan showing the arrangement of the piles and a typical cross-section through the foundation are shown in Fig. 8 and Fig. 9 respectively. Due to the various spacing of the piles, a part of wall is chosen for analysis which repeats along the length of wall. This part includes two piles in Row 1 and single piles in Rows 2 and 3 as shown. The loads to which this part of wall is subjected are as follows :

$V = -438\,k.$, $H = -81{\cdot}5\,k.$, $M = -16k.$ ft.

The moment M is evaluated about the P.G.C.

Equivalent ' area ' $= 3 \times 0{\cdot}9413 + 1 = 3{\cdot}8239$

Pile group centre $= \dfrac{3{\cdot}75 \times 0{\cdot}9413 + 7{\cdot}25}{3{\cdot}8239}$

$= 2{\cdot}8191$ from Row 1.

PILE NUMBERS			1	2	3	4
COEFFICIENTS OF *V*, *H* and *M*		*V*	— 0·5489	+ 0·2083	— 0·2319	— 0·4274
		H	+ 4·2022	— 6·2579	— 0·3901	+ 2·4466
		M	— 0·2719	+ 0·2519	+ 0·0462	— 0·0260
LOAD COMBINATION I	*V*	— 57·89	31·7758	— 12·0585	13·4247	24·7422
	H	— 11·06	— 46·4763	69·2124	4·3145	— 27·0594
	M	— 127·63	34·7026	— 32·1500	— 5·8965	3·3184
	TOTAL		20·0048	25·0039	11·8427	1·0012
LOAD COMBINATION II	*V*	— 44·90	24·6456	— 9·3527	10·4123	19·1903
	H	— 4·50	— 18·9099	28·1606	1·7555	— 11·0097
	M	— 71·04	19·3158	17·8950	— 3·2820	1·8470
	TOTAL		24·0515	0·9129	8·8859	10·0276
LOAD COMBINATION III	*V*	— 51·77	28·4166	— 10·7837	12·0055	22·1265
	H	+ 2·50	10·5055	— 15·6448	— 0·9753	6·1165
	M	+ 124·78	— 33·9277	31·4321	5·7648	— 3·2443
	TOTAL		4·9944	5·0036	16·7950	24·9987
LOAD COMBINATION IV	*V*	— 90·40	49·6206	— 18·8303	20·9638	38·6370
	H	— 5·15	— 21·6413	32·2282	2·0090	— 12·6000
	M	+ 41·34	— 11·2403	10·4135	1·9099	— 1·0748
	TOTAL		16·7390	23·8114	24·8827	24·9622
MAX. VERT. LOADS	KIPS.		25·0515	25·0039	24·8827	24·9987
MAX. AXIAL LOADS	KIPS.		25·8156	25·7666	24·8827	25·4959
MAX. AXIAL LOADS	TONS		11·5248	11·5029	11·1083	11·3821

TABLE 6.

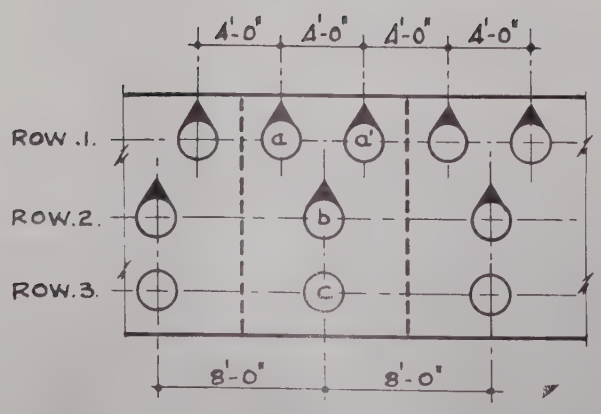


FIG. 8.

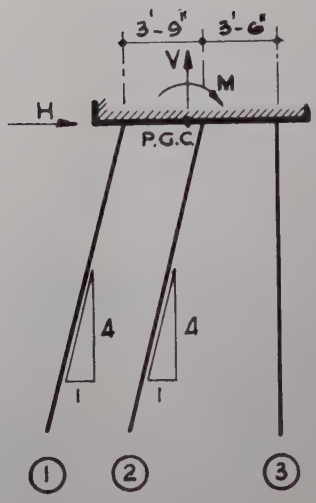


FIG. 9.

Pile No.	Vertical Loads F (kips)	Axial Loads on Piles $F/\cos \phi$ (kips)	Axial Loads on Piles $F/\cos \phi$ tons	Horizontal Forces $F \tan \phi$ (kips)	Moments About P.G.C. Fx k. ft.
a	108·76	112·08	50·04	27·19	+ 306·61
a′	108·76	112·08	50·04	27·19	306·61
b	108·49	111·80	49·91	27·12	— 100·99
c	111·99	111·99	50·00	0	— 496·23
Total	438·00			81·50	16·00

TABLE 7.

The elements of the ' flexibility matrix ' are :

$a_{11} = 3 \cdot 8239$

$a_{12} = a_{21} = 3 \times 0 \cdot 2353 = 0 \cdot 7059$

$a_{13} = a_{31} = 0 \qquad a_{22} = 3 \times 0 \cdot 0588 = 0 \cdot 1764$

$a_{23} = a_{32} = 2 \times 0 \cdot 2353 \times 2 \cdot 8191 - 0 \cdot 2353 \times 0 \cdot 9309 = 1 \cdot 1076$

$a_{33} = 0 \cdot 9413\,(2 \times 2 \cdot 8191^2 + 0 \cdot 9309^2) + 4 \cdot 4309^2 = 35 \cdot 4102$

$$-\begin{bmatrix} V \\ H \\ M \end{bmatrix} = \begin{bmatrix} \Delta_V \\ \Delta_H \\ \Delta_R \end{bmatrix} \begin{bmatrix} 3 \cdot 8239 & 0 \cdot 7059 & 0 \\ 0 \cdot 7059 & 0 \cdot 1764 & 1 \cdot 1076 \\ 0 & 1 \cdot 1076 & 35 \cdot 4102 \end{bmatrix}$$

Substituting for V, H and M.

$\Delta_V = 112 \cdot 3354, \quad \Delta_H = 11 \cdot 9585, \quad \tilde{\varepsilon} \quad \Delta_R = 0 \cdot 0776.$

The vertical loads on piles are given by :

$Fa = Fa' = 0 \cdot 9413\,\Delta_V + 0 \cdot 2353\,\Delta_H + 2 \cdot 6536\,\Delta_R$

$Fb = 0 \cdot 9413\,\Delta_V + 0 \cdot 2353\,\Delta_H - 0 \cdot 8763\,\Delta_R$

$Fc = \Delta_V - 4 \cdot 4309\,\Delta_R$

Substituting for Δ_V, Δ_H and Δ_R the vertical and axial loads on the piles have been evaluated and the statical equilibrium checked in Table 7.

ACKNOWLEDGEMENT

The author acknowledges the kind permission of R. Travers Morgan & Partners for the publication of the data contained in this paper.

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DISCUSSION

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by November 30th, 1960.

Book Review

Reinforced Concrete Reservoirs and Tanks, by W. S. Gray, 4th Edition., Revised by G. P. Manning. (London : Concrete Publications, 1960). 9 in. × 6 in., 190 pp. 12s.

For the fourth edition, this well-known book has been extensively revised to include up-to-date methods of the design and construction of most types of open and covered reservoirs, tanks on the ground and below ground, swimming pools, and such industrial structures as gasholder tanks, tar tanks, and the like. Special features include several charts and other data relating to the design of reinforced concrete works in accordance with the new British Standard Code of Practice No. 2007 (1960) for structures for the storage of water and other aqueous liquids. As in previous editions, Dr. Reissner's method of calculating the bending moments on the walls of cylindrical tanks is given in full. Numerous additional examples, with working drawings and illustrations showing the structures in the course of construction, are given.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1960-61 will be given by Lt.-Colonel G. W. Kirkland, M.B.E. (Mil.), M.I.Struct.E., M.I.C.E., at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 6th October, 1960, at 6 p.m.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 13th October, 1960.

Joint Meeting of the Institution of Structural Engineers and the British Section of the Société des Ingénieurs Civils de France at 6 p.m., when MM. Jean Bellier and Max Tourasse will read a paper entitled "Prestressed Concrete Pressure Vessels for Nuclear Reactors at Marcoule."

Thursday, 27th October, 1960.

Ordinary General Meeting at 5.55 p.m. for the election of members. This meeting, which is open only to corporate members of the Institution will be followed by an Ordinary Meeting at 6 p.m., when a paper entitled "Structural Considerations of a New Animal Feed Mill in Belfast, Northern Ireland" will be read by Mr. E. V. Finn, M.I.Struct.E., A.M.I.C.E. and Mr. K. F. Shadbolt, A.M.I.Struct.E.

Members wishing to bring guests to the Ordinary and Joint Meetings announced above are requested to apply to the Secretary for tickets of admission.

APPOINTMENT OF SECRETARY

Applications are invited for the position of Secretary of the Institution of Structural Engineers from men between the ages of 35 and 50. Applicants, who must be British, should possess a University degree, preferably with honours, or be a corporate member of a chartered engineering Institution, or have other suitable professional or service qualifications, and must have wide administrative experience.

Commencing salary will be £3,000 per annum; there is a contributory superannuation scheme.

The successful applicant will be required to take over from the retiring Secretary on the 1st October, 1961.

Further particulars may be obtained on application to The Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1., the envelope to be marked "Appointment of Secretary."

The closing date for the submission of the detailed applications will be 30th November, 1960.

EXAMINATIONS, JANUARY, 1961.

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 10th and 11th January, 1961 (Graduateship) and the 12th and 13th January, 1961 (Associate-Membership).

FORMATION OF A SOUTHERN BRANCH

Notice of a proposal to form a Southern Branch of the Institution was given in the June issue of the Journal. The Council has now sanctioned the establishment, according to the Bye-Laws, of a Southern Branch, comprising those members, corporate and non-corporate, resident in Dorset, that part of Wiltshire south of Salisbury Plain, Hampshire, the Isle of Wight and West Sussex.

The Chairman of the new Branch is Mr. J. J. Leeming, B.Sc., M.I.C.E. (Member) and the Honorary Secretary is Mr. A. P. K. Tate, B.Sc. (Associate-Member).

The opening meeting of the Branch will be held at Southampton on Friday, 4th November, 1960.

AWARD OF MERIT AND SCHOLARSHIP FUND

The Council has made an award of ten guineas each from the Award of Merit and Scholarship Fund to the following candidates in recognition of the high standard attained by them in the Institution's examinations for the Session 1958-1959:

Mr. Ho Tong Yam—Associate-Membership Examination

Mr. P. Head—Associate-Membership Examination

REPRESENTATION

The Council has appointed the following Institution representatives:

Building and Civil Engineering Advisory Committee (Birmingham College of Technology)

Mr. E. R. Deeley (Member) (re-appointment)

Building Advisory Committee (Brooklyn and Handsworth Technical Colleges and the Bordesley Green Technical Institute)

Mr. E. R. Deeley (Member) (re-appointment)

Bradford Technical College Advisory Committee on Building

Mr. F. A. Maddock (Member)

British Standards Institution Codes of Practice, Scottish Advisory Committee CP/-/2

Mr. A. Mason (Member)

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A candidate for Graduateship or Associate-Membership may be able to attend a Technical College; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION—

Technical Colleges offer:

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the Candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At Technical Colleges, courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and

Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some Technical Colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering. These cover only part of the requirements for the Associate-Membership Examination.

Colleges in List 'A' provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

LIST 'A'

Bath Technical College.
Belfast College of Technology.
Birmingham College of Advanced Technology.
Bolton Technical College.
Bradford Institute of Technology.
Bridgend Technical College.
Bristol College of Technology.
Chesterfield College of Technology.
Coatbridge Technical College, Lanarkshire.
Derby and District College of Technology.
Dudley and Staffordshire Technical College.
Ewell County Technical College.
Glasgow Royal College of Science and Technology.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith College of Art and Building.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Portsmouth College of Technology.
Salford, Royal Technical College.
South-East London Technical College, Worseley Bridge, S.E.26.
South-West Essex Technical College, Walthamstow, E.17.
Southampton Technical College.
Stafford County Technical College.
Stockport College of Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List 'B' provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates, is not usually so complete.

LIST 'B'

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea College of Technology, S.W.11.
London, Northampton College of Advanced Technology, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle upon Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston Harris Institute.
Rotherham College of Technology.
S.E. Essex Technical College, Dagenham.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

ADDITIONS TO THE LIBRARY

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Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The opening meeting of the Session will be held on Tuesday, 18th October, 1960, at the College of Science and Technology, Manchester, commencing at 7.0 p.m., when Professor W. Merchant, M.A., S.M., D.Sc., M.I.Struct.E., M.I.C.E., A.M.I.Mech.E., will give the Chairman's Address.

The meeting will be preceded by a Hot Pot Supper. A coach will run from Liverpool.

Hon. Secretary : W. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs.

MIDLAND COUNTIES BRANCH

The Annual Dinner and Ladies' Evening will be held at the Botanical Gardens, Birmingham, on Saturday, 15th October, 1960.

A Meeting of the Branch will be held at the Byng Kendrick Suite, College of Advanced Technology, Gosta Green, Birmingham, 6., on Friday, 28th October, 1960, at 6.30 p.m., when the Chairman's Address will be given by Mr. G. Botteley, T.D., M.I.Struct.E. Tea will be served from 5.45 p.m.

Hon. Secretary: S. M. Cooper, A.M.I.Struct.E., "Applegarth," Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

The first meeting of the Session will be held at the Engineering Centre, Stephenson Place, Birmingham, and will start at 6.30 p.m. on Friday, 4th November, 1960. A paper will be read by Mr. C. R. Blackwell, of Freeman, Fox and Partners, entitled "Design and Construction of the Volta Bridge." Tea will be served from 5.30 p.m.

Hon. Secretary: H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

A meeting will be held in the Neville Hall, Newcastle, at 6.30 p.m., preceded by a buffet tea from 6.0 p.m., on Wednesday, 5th October, 1960, when Mr. E. Ibbotson, M.I.Struct.E., will be installed as Chairman, and give the Chairman's Address.

On Monday, 17th October, 1960, installation of Branch Chairman by the Institution President-Elect., Lt.-Col. G. W. Kirkland, M.B.E. (Mil.), M.I.Struct.E., M.I.C.E., Chairman's Address by Mr. E. Ibbotson, M.I.Struct.E., at 6.30 p.m. preceded by a buffet tea from 6.0 p.m.

Hon. Secretary: P. D. Newton, B.Sc., M.I.Struct.E., A.M.I.C.E., c/o Cussson and Partners, 112, Borough Road, Middlesbrough, Yorkshire.

NORTHERN IRELAND BRANCH

The opening meeting of the Session will be held on Tuesday, 4th October, 1960, in the Civil Engineering Department, David Keir Building, Queen's University, Belfast, at 6.30 p.m. Tea will be served from 5.45 p.m. to 6.30 p.m. The Chairman's Address will be given by Mr. E. A. Plester, M.I.Struct.E.

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, 10th October, 1960, at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. The Chairman's Address will be given by Mr. N. M. Brydon, M.B.E., B.Sc., M.I.Struct.E., M.I.C.E.

The Annual Dinner and Dance will be held on Tuesday, 11th October, 1960, at the Grosvenor Restaurant, Gordon Street, Glasgow, at 6.30 p.m.

Hon. Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTH WESTERN COUNTIES SECTION

Hon. Secretary: C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

SOUTHERN BRANCH

The Inaugural Meeting of the Southern Branch was held at the University of Southampton on the 22nd July, 1960. The President of the Institution, Mr. L. E. Kent, installed Mr. J. J. Leeming as first Chairman of the Branch and Mr. Leeming then addressed the meeting.

The following Honorary Officers and members of the Committee have been elected for the Session 1960-61 :

Hon. Secretary : A. P. K. Tate, B.Sc., A.M.I.Struct.E., South Stoneham House, Swaythling, Southampton.

Hon. Treasurer : C. Carter.

Committee : G. J. Gumersall, J. A. Harvey J. R. Lowe, Professor P. B. Morice, J. H. Morris, Leslie Turner, H. V. Williams.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorganshire.

WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held on Friday, 7th October, 1960, at the University Engineering Laboratories, Bristol, at 6 p.m., preceded by tea at 5.30 p.m.

The Chairman's Address will be given by Mr. Clifford E. Saunders, M.I.Struct.E.

Hon. Secretary: A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

YORKSHIRE BRANCH

The opening meeting of the Session will be held on Wednesday, 19th October, 1960, at the Griffin Hotel, Leeds, when Dr. W. Eastwood will give the Chairman's Address. This will be repeated on Thursday, 20th October, at the Royal Victoria Hotel, Sheffield. Both meetings will commence at 6.30 p.m. preceded by buffet tea at 6.15 p.m.

Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorkshire.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: E. B. Kretzschmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary : W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown and Co. Ltd., P.O. Box 648, Singapore.

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Hon. Secretary : A. Brimer, A.M.I.Struct.E., Brimer, Andrews and Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

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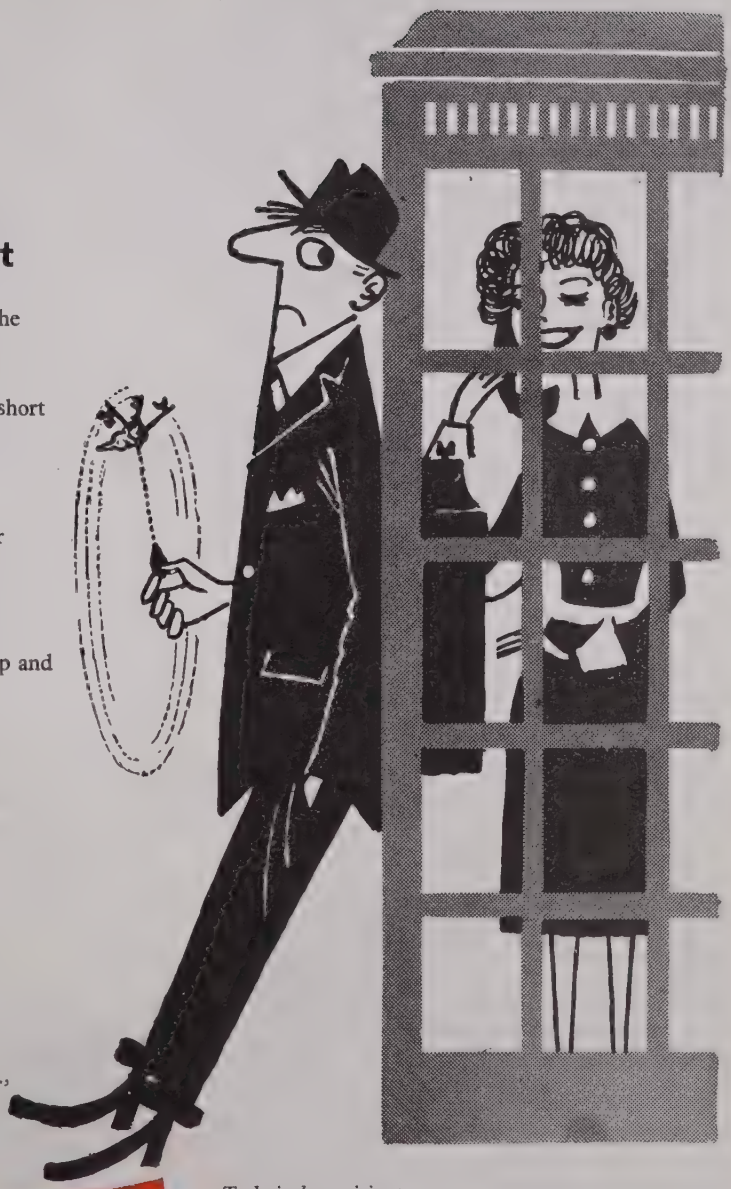

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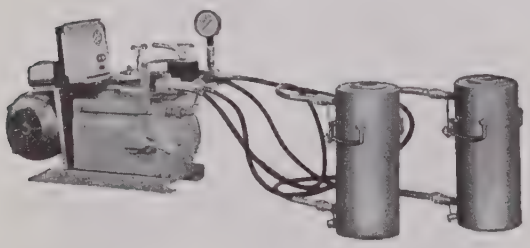

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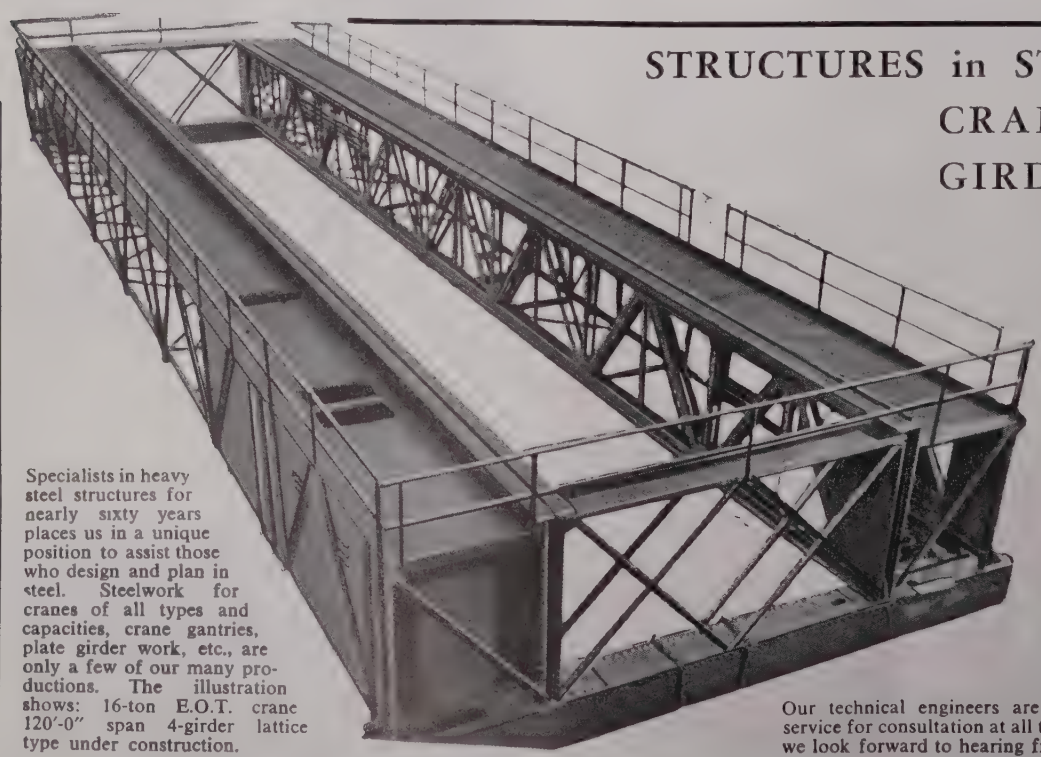

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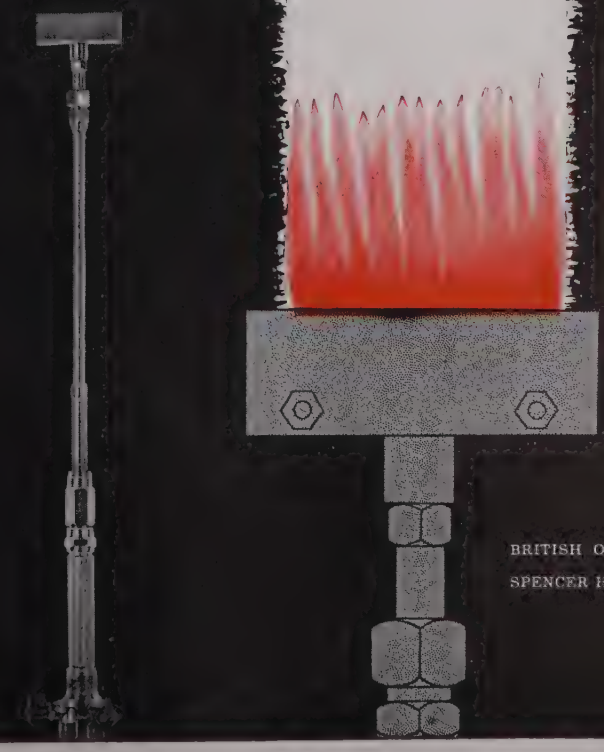

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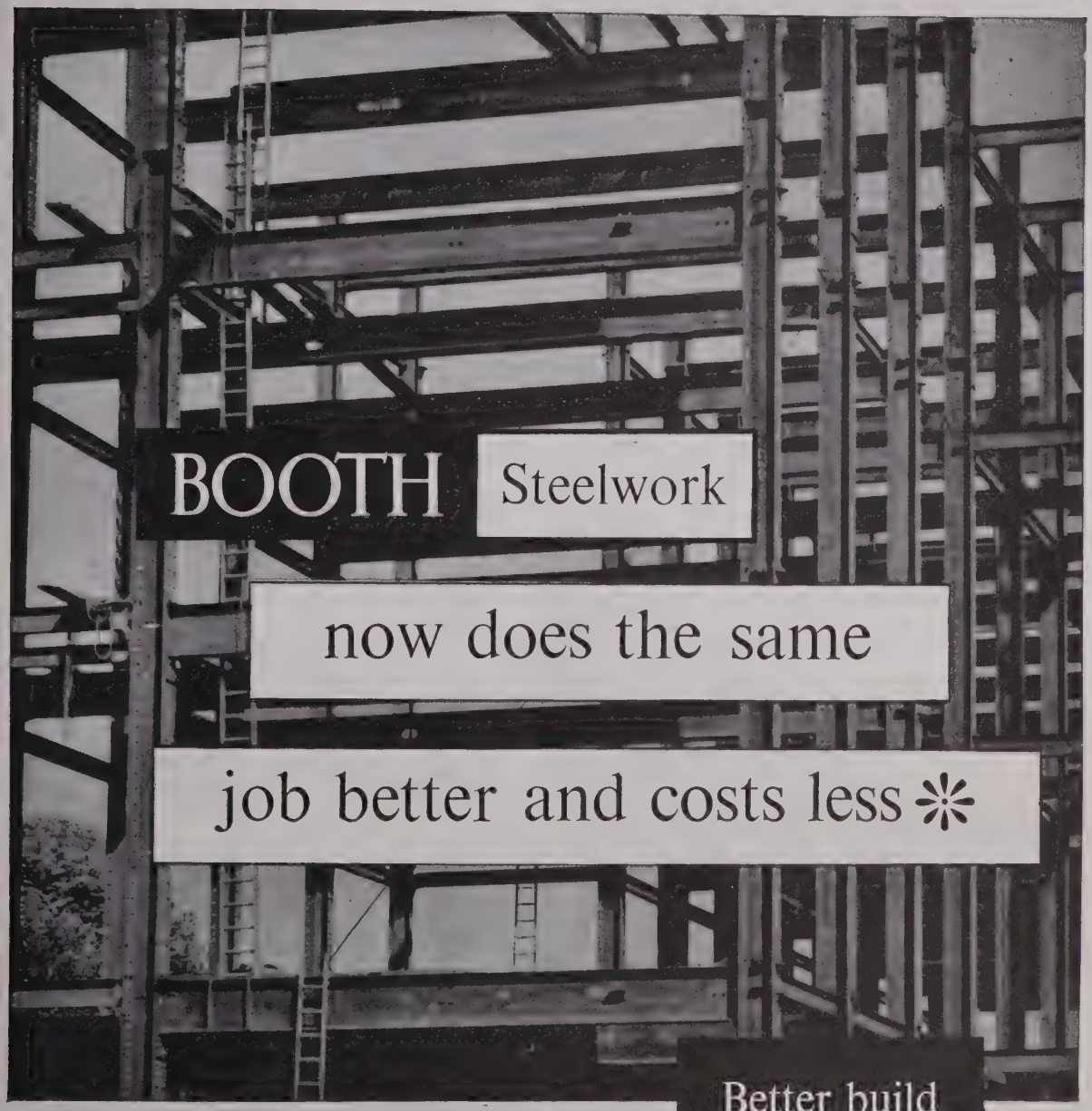

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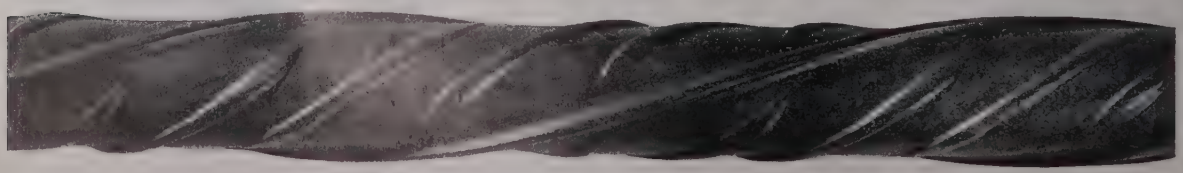

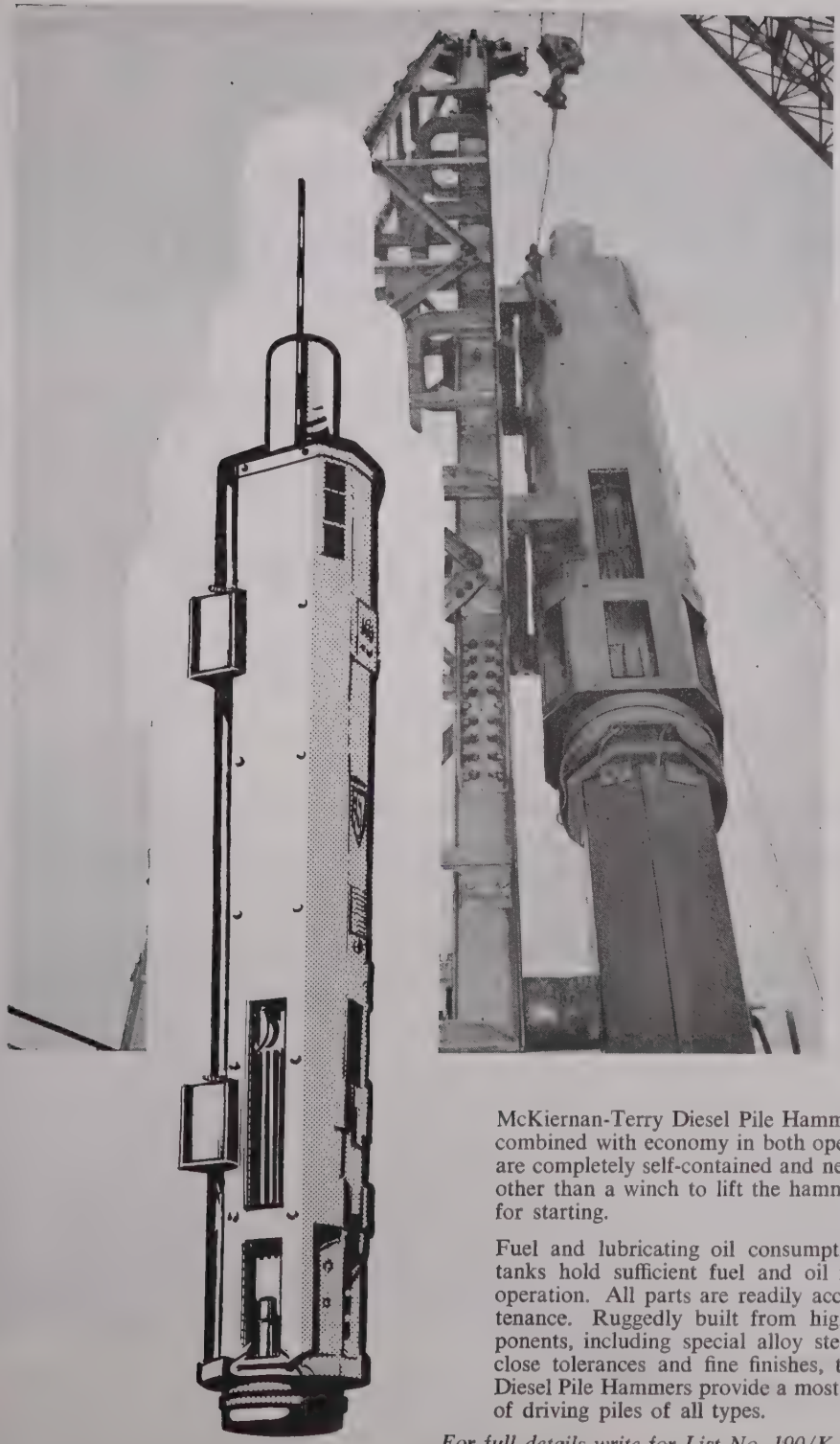

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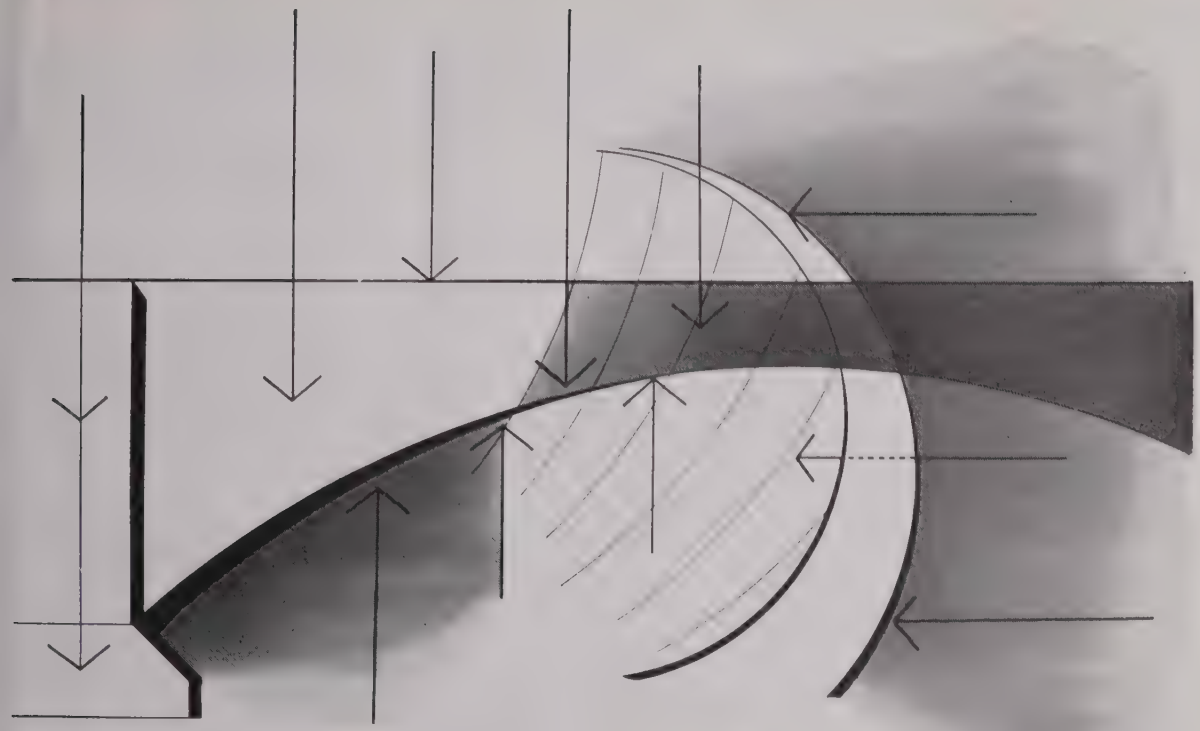

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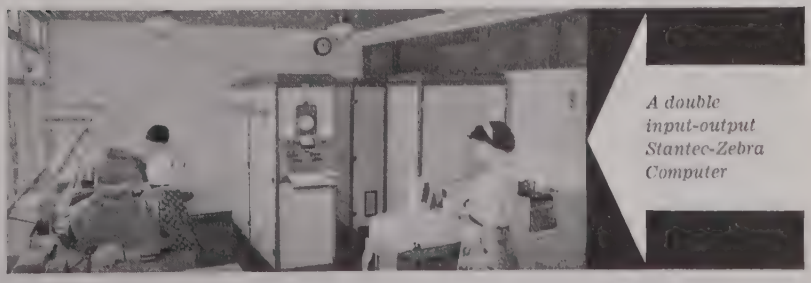

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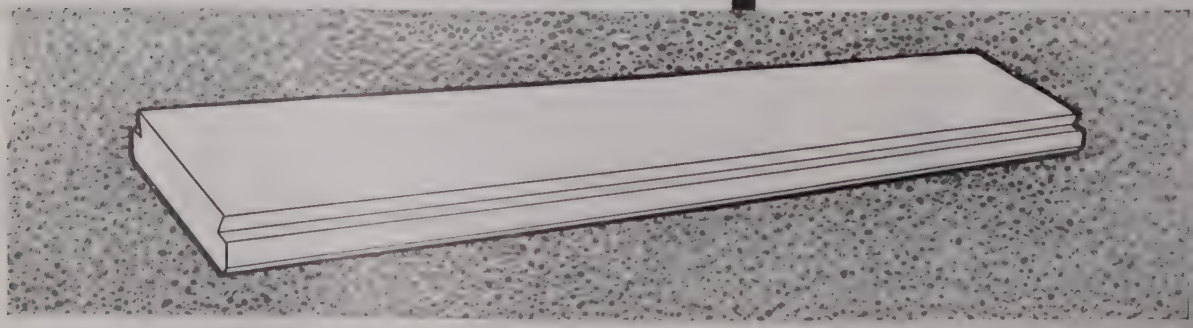

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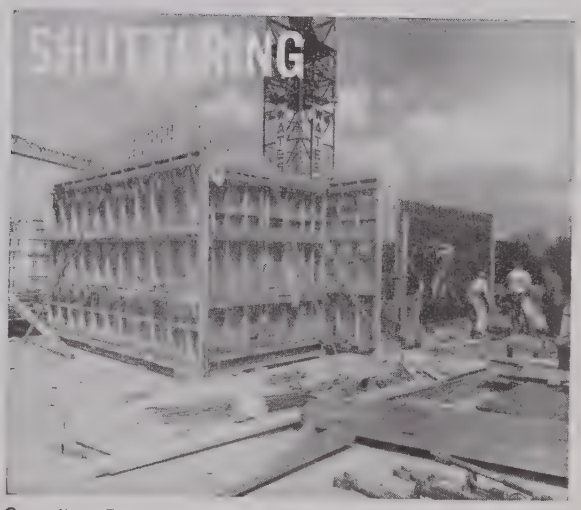

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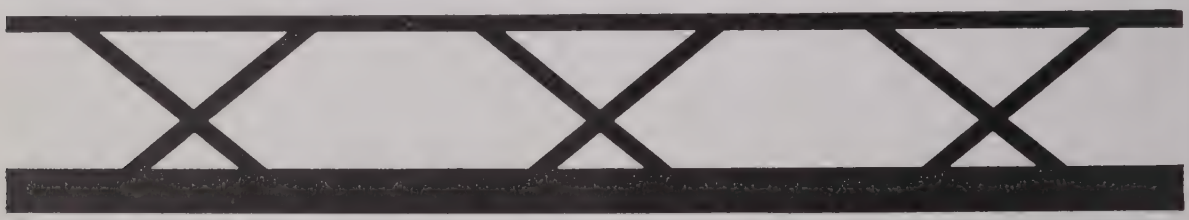

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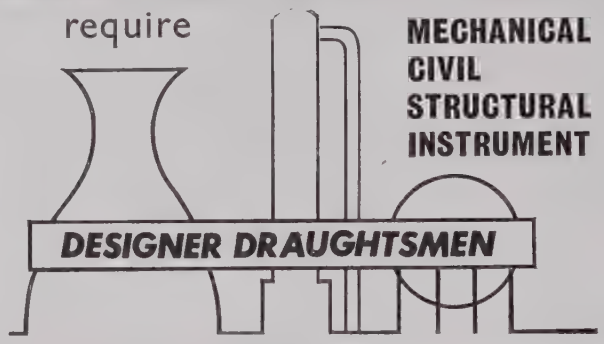

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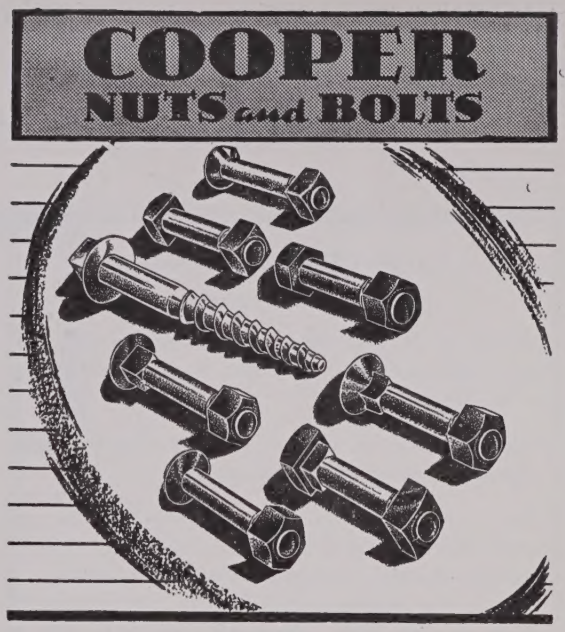

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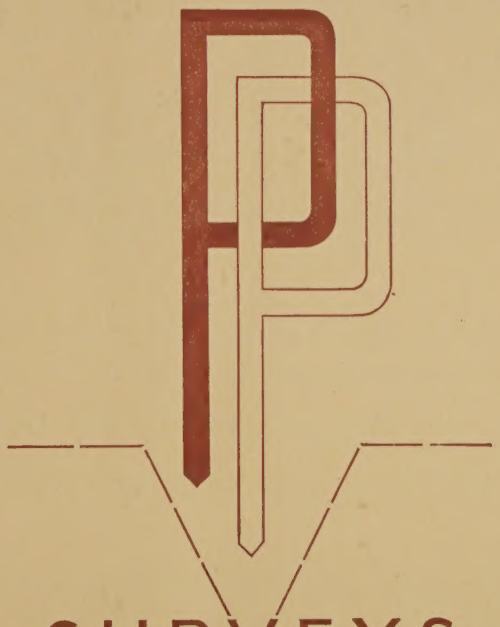

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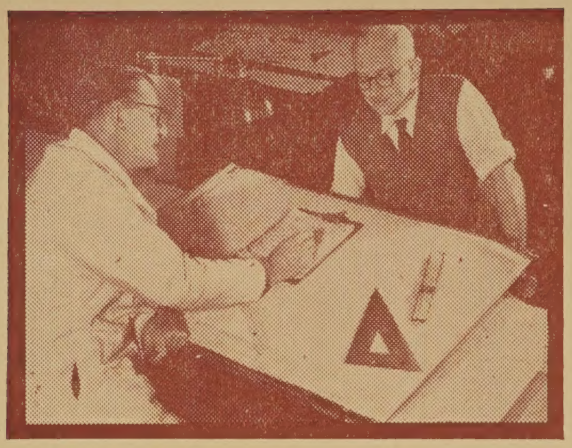

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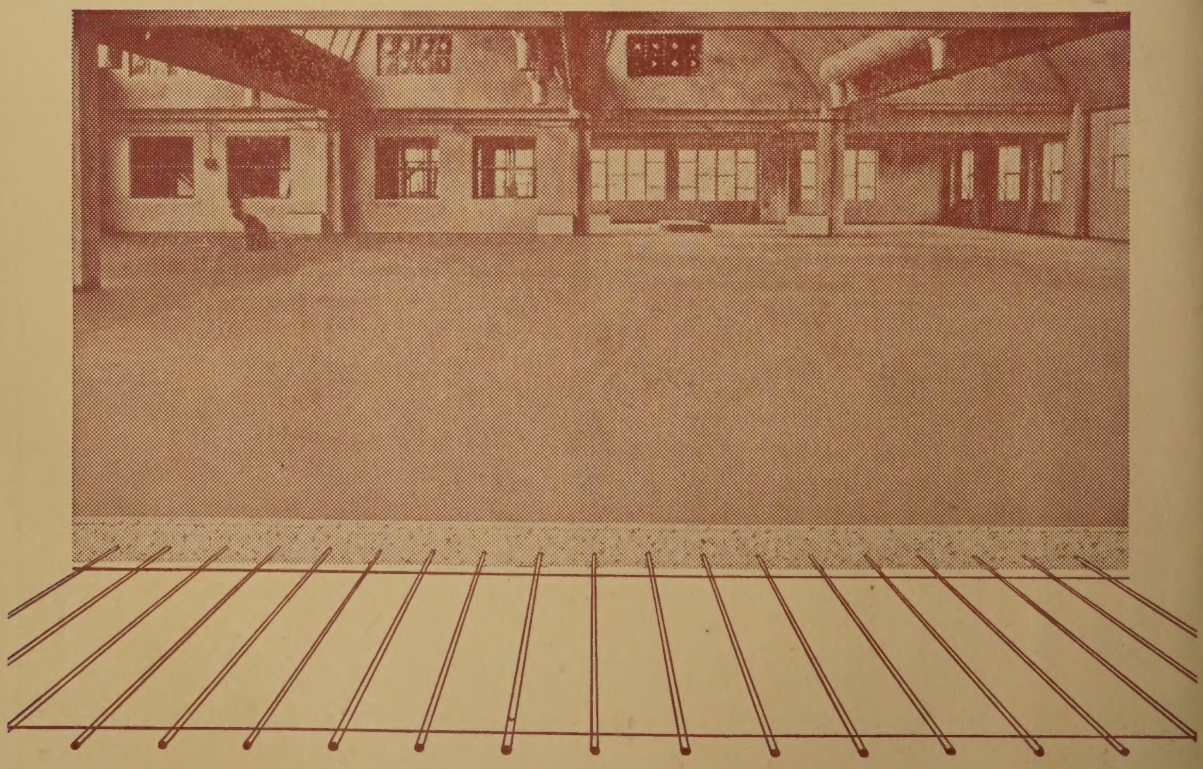

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